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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

IMPROVEMENT OF HUDSON RIVER BY NARROWING THE NAVIGABLE FAIRWAY

By Lynne J. Bevan,[1] M. Am. Soc. C. E.

Synopsis

This paper applies to the problem of pier extensions in the Hudson River west of the Island of Manhattan, the broad hydraulic principles of waterway design. It presents data indicating that successive encroachments on the navigable fairway by the extension of piers have set in action natural forces which have scoured the river bottom, actually increased the cross-section area at a substantial saving in dredging expense, and, contrary to generally accepted statements, have actually decreased tidal current velocities.

Narrowing a broad natural waterway and simultaneously increasing its depth tends to improve the hydraulic regimen of the waterway; it decreases the tendency to create cross-currents, increases the hydraulic radius, decreases the mean velocity necessary for a definite quantity of water, and tends to make velocity more uniform from the surface to the bottom.

Numerous illustrations of the general acceptance of this hydraulic principle are recorded in technical publications. An impressive illustration of its application occurs at the mouth of the Columbia River, on the Pacific Coast, where two jetties north and south, have concentrated the tidal flow, so that to-day (1932) a single 40-ft. channel 1 mile wide is easily maintained, whereas the old natural channels, usually less than 20 ft. deep, wandered aimlessly over a 7-mile front.

Owing to conflicting interests, real and imaginary, as between the States of New York and New Jersey, between trans-oceanic and local commerce, between economics and politics, and between various other elements, efforts to apply this principle to the Hudson River, by advancing pierhead lines from both the New York and the New Jersey shores, have always met with opposition.

Note.—Discussion of this paper will be closed in September, 1932, *Proceedings*.

[1] Cons. Engr., New York, N. Y.

In 1767, the shore lines of the Hudson opposite Lower Manhattan were approximately twice as far apart as are the pierhead lines in 1932. In 1857, a pierhead line on the Manhattan shore was established by the United States War Department, and since that date several lines farther from both the New York and New Jersey shores have been approved by that Department. These advances usually have been requested by trans-oceanic steamship interests that required longer piers at which to dock vessels of ever-increasing lengths. They have been opposed by towboat companies and other interests that have advocated the maintenance of the wide-water areas with which

TABLE 1.—COMPARISON OF CROSS-SECTION AREAS OF THE HUDSON RIVER, IN SQUARE FEET, BETWEEN PIERHEAD LINES AS ESTABLISHED IN 1930 AND AS ESTABLISHED IN THE YEARS SHOWN

Description	1845	1855	1874	1882	1912	1930
Barclay Street Section:						
Between pierhead lines in year named	....	....	154 398	139 260	142 940	141 900
Between 1930 pierheads; section in year named	128 554	129 585	143 782	132 369	142 940	141 900
Chelsea Piers (18th Street):						
Between pierhead lines in year named	....	....	144 116	149 915	159 979	159 500
Between 1930 pierheads; section in year named	128 798	139 332	142 591	148 882	159 979	159 500
Typical sections at 39th Street, 41st Street, and 52d Street:						
Between pierhead lines in year named	....	....	125 728	123 446	120 695	123 200
Between 1930 pierheads; section in year named	....	....	114 428	112 746	120 695	123 200

Nature has bountifully endowed New York Harbor. This opposition has averred that pierhead line advances have increased greatly the velocity of the tidal currents, and, hence, have handicapped navigation.

The data correlated in this paper were assembled with the hope of developing the effect of pierhead line, advances on the hydraulics of the river. Cross-

FIG 1.—KEY MAP TO SECTION NUMBERS

section areas (in square feet) between pierhead lines at various points along the west shore of Manhattan are compared in Table 1. Their locations are shown in Fig. 1 and the Chelsea Sections, plotted in Fig. 2, are illustrative.

These cross-sections show that in the mid-Manhattan area natural forces have compensated for the advance of pierhead lines by scouring of the river bottom. For instance, in the Chelsea Sections, where the last pierhead line modification occurred in 1897, the cross-section area in 1874 was 144 116 sq. ft. between the pierhead lines of that date, and, in 1912, the cross-section

area was 159 979 sq. ft. between the pierhead lines as then fixed, and this area was maintained until 1930.

The second series consists of cross-sections at eighteen, approximately equi-distant, locations between Governors Island and Fort Washington, for 1855, 1910, and 1930 (see Fig. 1). The sections for 1855 and 1910 were traced from a map dated November 28, 1922, entitled "Sections Showing Comparison of Channel Depths of 1855 and 1910" in the United States Engineer Office, First New York District, and the sections for 1930 were plotted from the 1930 edition of the U. S. Coast and Geodetic Survey Chart. The rough location of these sections is indicated in Fig. 1 and the defining limits in each case are approximately, as given in Table 2.

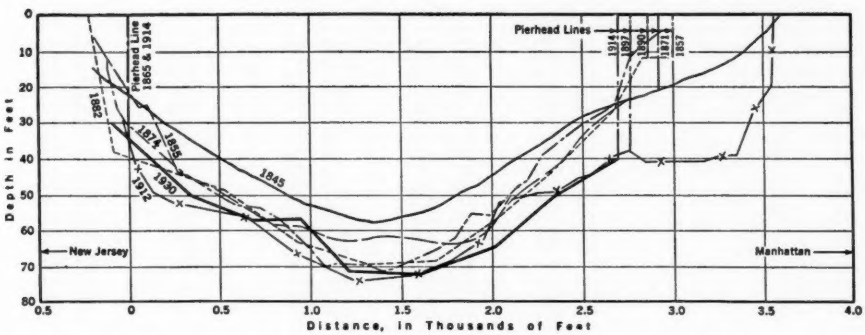

FIG. 2.—CROSS-SECTIONS OF CHANNEL OF THE HUDSON RIVER IN THE VICINITY OF THE CHELSEA PIERS (18TH STREET)

TABLE 2.—LOCATION OF SECTIONS SHOWN IN FIG. 1

Section No.	From New York City	To New Jersey	Section No.	From New York City	To New Jersey
1....	Governors Island...	Ellis Island	9....	West 52d Street....	West Shore Terminal, Weehawken
2....	Pier 1, the Battery..	Communipaw			
3....	Pier 14, Fulton Street	Pennsylvania Railroad Ferry, Jersey City	10....	West 61st Street...	10th Street, West New York
			11....	West 77th Street...	Guttenberg
4....	Desbrosses Street Ferry...........	Erie Ferry, Jersey City	12....	West 89th Street...	North Bergen
			13....	West 101st Street..	Shady Side, N. Bergen
5....	West 10th Street...	Delaware Lackawanna and Western Ferry, Hoboken	14....	West 113th Street..	New York Susquehanna and Western Railroad, Edgewater
6....	West 15th Street...	Castle Point, Hoboken	15....	West 125th Street..	Edgewater
7....	West 25th Street...	Weehawken Cave	16....	West 138th Street..	Fort Lee
8....	West 37th Street...	Delaware and Hudson Basin, Weehawken	17....	West 155th Street..	Fort Lee
			18....	Fort Washington Point............	Fort Lee

Four typical sections are graphically shown in Figs. 3, 4, 5, and 6. The areas are listed in Tables 3 and 4. Column (6), in each case, shows the actual 1930 areas while Column (4) indicates the part of the total area for which natural forces are responsible. It is obtained by deducting the area dredged (Column (5)) from the total 1930 area.

The cross-section areas listed in Tables 3 and 4 show, with greater detail, that natural forces have compensated for the advances of pierhead lines. It is a simple corollary of increased cross-section areas, from correspondingly

TABLE 3.—HUDSON RIVER CROSS-SECTION AREAS, IN SQUARE FEET, MEASURED TO PIERHEAD LINES AS OF THE DATES OF THE COLUMN HEADINGS

Section No. (1)	1855 (2)	1910 (3)	By Nature, 1930 (4)	Dredged (5)	Total, 1930 (6)
1	222 800	251 000	265 300	700	266 000
2	145 000	148 000	143 200	7 600	150 800
3	140 300	135 500	133 900	7 600	141 500
4	140 800	137 400	132 100	100	132 200
5	150 000	144 300	146 200	300	146 500
6	133 000	136 700	157 300	2 200	159 500
7	117 000	118 000	128 700	4 300	133 000
8	107 800	114 400	119 500	......	119 500
9	121 000	120 400	124 800	......	124 800
10	121 600	114 000	109 800	9 700	119 500
11	116 200	113 000	116 300	1 800	118 100
12	115 800	114 500	117 200	500	117 700
13	118 900	109 000	111 000	3 200	114 200
14	100 800	120 000	120 000	3 500	123 500
15	120 200	113 000	112 600	4 600	117 200
16	114 900	104 000	107 600	......	107 600
17	129 000	120 500	115 100	......	115 100
18	135 500	144 300	159 500	......	159 500
Mean	130 590	131 010	134 450	2 560	137 010

increased depth by scouring by natural forces, that substantial costs of dredging have been saved.

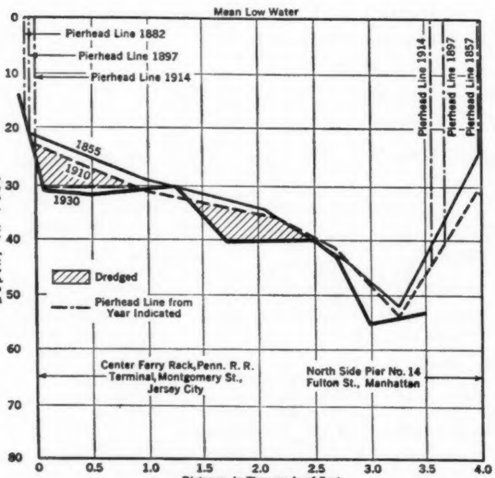

FIG. 3.—COMPARATIVE DEPTHS IN SECTION 3

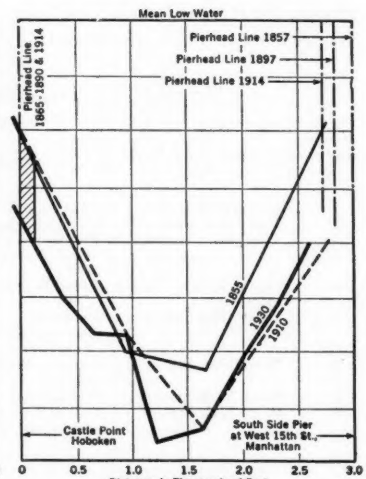

FIG. 4.—COMPARATIVE DEPTHS IN SECTION 6

A third series of six cross-sections was prepared in order to learn the effect, if any, of pierhead line advances upon the channel up stream from New York. These sections illustrate the differences in the channel bottom off

Haverstraw, N. Y., 36 miles north of the Battery, in 1916 and 1927 (see Fig. 7). As illustrative of this series, Figs. 8, 9, and 10 show typical profiles. The section areas were not planimeterd; but from the diagrams it is evident that Nature is maintaining these up-stream channels, and that no injury is resulting from pier extensions.

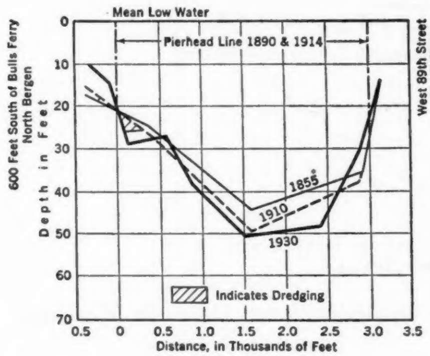

FIG. 5.—COMPARATIVE DEPTHS IN SECTION 12

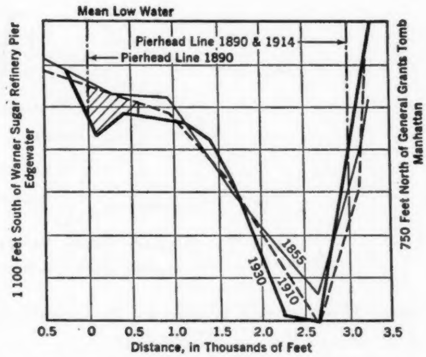

FIG. 6.—COMPARATIVE DEPTHS IN SECTION 15

These sections supplement the conclusion of the New York Harbor Line Board which, in 1911, compared cross-sections of the Hudson River at Yonkers, Fort Montgomery, and Poughkeepsie, as of 1955-61, with cross-sections at the same points as of 1903-05. After finding "practically no change," the Board concluded that it was fair to assume that no injurious

TABLE 4.—HUDSON RIVER CROSS-SECTION AREAS, IN SQUARE FEET, MEASURED TO THE 1930 PIERHEAD LINES

Section No. (1)	1855 (2)	1910 (3)	By Nature, 1930 (4)	Dredged (5)	Total, 1930 (6)
1	223 000	251 000	265 300	700	266 000
2	140 800	143 500	143 200	7 600	150 800
3	122 400	126 200	133 900	7 600	141 500
4	118 100	128 400	132 100	100	132 200
5	127 300	135 000	146 200	300	146 500
6	128 300	146 200	157 300	2 200	159 500
7	104 300	118 100	128 700	4 300	133 000
8	91 500	114 400	119 500	.....	119 500
9	103 500	120 300	124 800	.....	124 800
10	106 900	113 900	109 800	9 700	119 500
11	97 100	113 000	116 300	1 800	118 100
12	104 000	114 400	117 200	500	117 700
13	102 000	109 000	111 000	3 200	114 200
14	93 600	120 000	120 000	3 500	123 500
15	104 900	113 200	112 600	4 600	117 200
16	93 600	103 900	107 600	.....	107 600
17	103 700	106 500	115 100	.....	115 100
18	133 600	144 300	159 500	.....	159 500
Mean	116 590	128 960	134 450	2 560	137 010

effects on the river to the north of the city limits were found, that can be ascribed with certainty to the extension of harbor lines in New York.

Tidal currents, during the period covered by these cross-sections (1855 to 1930), have remained substantially constant. This statement is based upon

the common understanding that tidal currents exhibit changes in the strength of the current that correspond closely with the range exhibited by tides. As indicative of constancy in tidal range, the values for the tide at Yonkers,

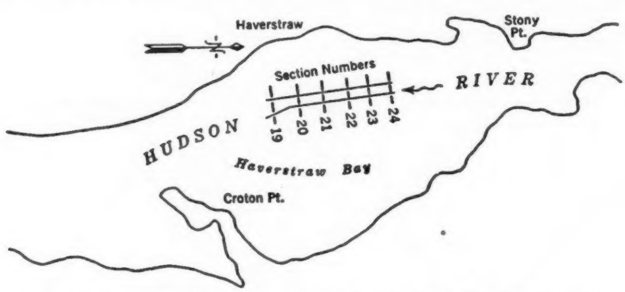

FIG. 7.—HUDSON RIVER IN VICINITY OF HAVERSTRAW, N. Y.
KEY MAP TO SECTION NUMBERS

from the records of the U. S. Coast and Geodetic Survey Office, in Washington, are herewith tabulated:

August 5 to 25, 1853...... 3.94 ft...Compared with Governors Island
July 16 to 27, 1855........ 4.08 ft...Compared with Governors Island
August 13 to November 14,
1898 .... 3.77 ft...Compared with Fort Hamilton

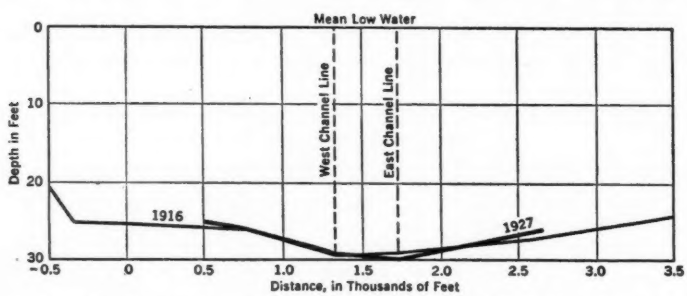

FIG. 8.—COMPARATIVE DEPTHS IN SECTION 20, HAVERSTRAW, N. Y.

Table 5 shows the ranges derived from observations for three months (July to September) of each of the years indicated and corrected for longitude, or moon's node.

TABLE 5.—TIDAL RANGE AT YONKERS, N. Y.

Year	Range, in feet	Year	Range, in feet	Year	Range, in feet
1910	3.40	1917	3.47	1924	3.42
1911	3.33	1918	3.35	1925	3.52
1912	3.19	1919	3.52	1926	3.52
1913	3.33	1921	3.48	1927	3.48
1914	3.32	1922	3.57	1928	3.61
1915	3.40	1923	3.42	1929	3.57
1916	3.35	....	....	....	....

The New York Harbor Line Board investigation of 1911, found also that there had been practically no change in the rate of progress of the tidal wave between Governors Island and Tivoli (16 miles below Hudson, N. Y.) since 1855, and no change in the tidal range.

The Metropolitan Sewerage Commission (of New York City), consisting of five eminent engineers made extensive studies of tides and tidal currents

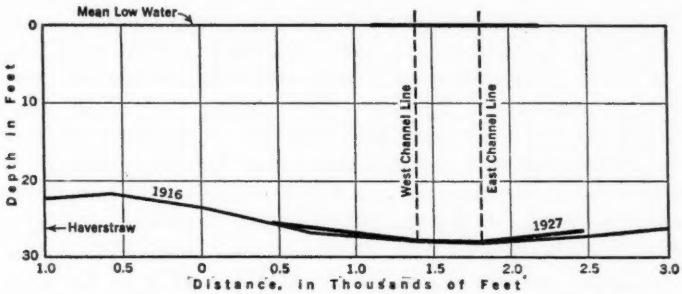

FIG. 9.—COMPARATIVE DEPTHS IN SECTION 22, HAVERSTRAW, N. Y.

in New York Harbor, and submitted a voluminous report under date of April 30, 1910. The following is quoted from the report (page 181):

"The extension of piers into rivers, especially the Hudson River would reduce the amount of tide water passing up and down. However, neither the reclamation of shore areas nor the extension of piers into the Hudson seem, up to the present time, to have sensibly interfered with the tide. For instance, the mean range of tide at Dobbs Ferry, determined by observations in the years 1856, 1858, 1885, 1886, and 1900 has the values of 3.71, 3.69, 3.58, 3.60 and 3.66 feet, respectively, and these figures are apparently sufficiently close to cover yearly variations."

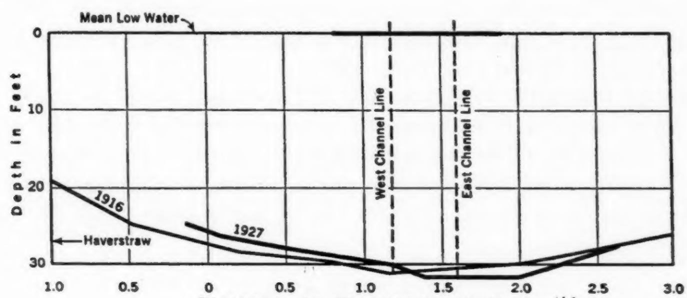

FIG. 10.—COMPARATIVE DEPTHS IN SECTION 24, HAVERSTRAW, N. Y.

The paradox between the first sentence of this quotation and the findings in the following sentences is doubtless due to the fact that the Hudson River bottom is of fine material, readily susceptible to scouring action of temporarily slightly increased velocities, and, once the scouring starts, it proceeds somewhat beyond the previous point of stabilization. The greater cross-section areas may be considered reasonably permanent because the narrowing

of the channel has improved the uniformity of currents from the surface to the bed of the river.

To an engineer experienced in stream-flow measurement the tremendous difficulty in obtaining reliable tidal current-velocity measurements in the Hudson River are readily appreciated. However, such measurements opposite Manhattan have been compiled and briefly discussed by H. A. Marmer, M. Am. Soc. C. E., in a paper[2] entitled, "Tides and Currents in New York Harbor." A cursory examination of the data on these currents, and particularly his Table 63, shows how inadequate available information is for drawing the conclusion that pier extensions have caused any increase in current velocity between 1854 and 1922. In this series covering sixty-seven stations it is seen that the velocities recorded for flood tides vary from 0.7 knot to 1.9 knots for the 1854 to 1885 measurements, and from 0.1 knot to 2.1 knots for the 1919 to 1922 measurements, and that the velocities for ebb tides vary from 1.0 knot to 2.8 knots for the 1854 to 1885 measurements and from 0.9 knot to 2.6 knots for the 1919 to 1922 measurements. (These last figures omit the record at one station, which Mr. Marmer states is the result of the use of a tidal range correction factor which is undoubtedly too large.)

The fact too rarely realized that stream flow measurements require tremendously greater skill, time for observations, and experienced judgment than soundings, may explain the ready acceptance of a few early flow measurements as a basis for the assumption of increase in tidal current velocities in the Hudson, as was done in some of the early reports on this subject.

The conclusion of the data assembled is that, inasmuch as the cross-section areas of the Hudson River between the Battery and Fort Washington have increased substantially between 1855 and 1930, and the tidal flow as shown by the mean range of tide at observation stations a short distance above Manhattan has remained substantially constant during this period, the mean tidal current velocity must necessarily have decreased. In other words, pier extensions into the Hudson River have generally decreased current velocities opposite Manhattan, since 1857.

The broad hydraulic principles which have been found to apply in the Hudson since pierhead lines began to advance into the river will doubtless continue to function. The tunnels will set a practical limit to increases of depth by scouring; but it is probable that a decrease in width of the navigable fairway by 10%, or possibly 20%, will not militate against favorable hydraulic conditions.

Acknowledgments

Acknowledgment for assistance in assembling material for this paper is gratefully made to Robert A. Lesler, and H. A. Marmer, Members, Am. Soc. C. E., and to the officers and civilian engineers in the U. S. Army Engineer Office, First District, New York. Most of the data in this paper were obtained from maps and records in that Office and from the U. S. Coast and Geodetic Survey Chart for 1930.

[2] *Special Publication No. 111,* Serial No. 285, U. S. Coast and Geodetic Survey, p. 124 *et seq.*

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS

APPLICATION OF DURATION CURVES TO HYDRO-ELECTRIC STUDIES

By G. H. Hickox[1] and G. O. Wessenauer,[2] Juniors, Am. Soc. C. E.

Synopsis

This paper outlines an accurate method of determining, from duration curves, the reliable power rate, and plant and reservoir capacities for a combination of hydro-electric plants and reservoirs on streams with similar flow distribution, but not necessarily in the same basin.

The method can be utilized for these problems only by studying data for exactly one reservoir cycle. This procedure is fundamental, and may have been overlooked by other investigators; at least, no study has been found in which this factor has been emphasized.

The use of the duration curve method for hydro-electric studies is explained in this paper by applying it to some simple hypothetical combinations of hydro-electric plants. Then, for an actual proposed hydro-electric combination, the results obtained by the use of the curves are compared with those obtained by analytical week-by-week studies (analyzed by days where necessary).

Introduction

Investigation of the power possibilities of Cheat River, by the West Virginia Power and Transmission Company, indicated the desirability of a rapid, fairly accurate method of determining the reliable power rate, and plant and reservoir capacities to be expected from various plant combinations.

Lake Lynn, a 50 000-kw., run-of-river, hydro-electric plant, is 3.5 miles up stream from the mouth of Cheat River, near Morgantown, W. Va., and eventually there may be about a dozen plants in operation in that river basin. The question of determining the order, capacity, and storage for succeeding plant installations gives rise to a series of problems that are difficult to solve by existing methods.

Note.—Discussion of this paper will be closed in September, 1932, *Proceedings*.

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Previous studies indicated that the next hydro-electric construction after the Lake Lynn plant should be a storage development above two high-head plants on the Blackwater River. The storage reservoir should have not only sufficient capacity to equalize and control its tributary stream flow and two-plant output, but also enough to aid in maintaining a high uniform power rate when acting in combination with the Lake Lynn plant. The problems of determining the combined dependable output from the Lake Lynn and Blackwater plants, and also the proper installed capacity for the latter, are complicated by the fact that occasionally potential energy drawn from storage may not be developed at the Lake Lynn plant. The occasional failure of storage water to create energy at this plant is due to the fact that some of the water would reach that point when the plant would be working at its maximum efficient capacity. A study of flow and resulting energy, based on monthly data, fails to reveal the extent of this loss. Accordingly, several analytical studies were made on a weekly flow basis, with detailed study of each day when the Lake Lynn plant would be working at, or close to, its maximum efficient capacity. The analytical studies thus became essentially day-by-day studies and involved excessive work. Therefore, it was obvious that some time could be spent profitably in devising a method which would expedite the solution of the complicated problems involved in the still later developments.

The two well-known methods of solving hydro-electric problems are the hydrograph and the mass curve. Some investigators have used duration curves for solving certain phases of hydro-electric problems, but no evidence has been found to indicate that such curves have been used to the extent outlined herein. In studies of power development in Northern West Virginia, F. W. Scheidenhelm and R. M. Riegel, Members, Am. Soc. C. E., used the duration curve in a manner that approaches more closely the method outlined in this paper than any others which could be found in a search of engineering literature.

This paper indicates that when the duration-curve method is applied to data for a complete reservoir cycle, it gives quick, accurate determinations of maximum rate of output, total output, division of output, and plant and reservoir capacities. Furthermore, a comparison indicates nearly as high a degree of accuracy for the results of the duration-curve method as for the more tedious hydrograph method.

Nomenclature

Terms which might be misunderstood are defined, as follows:

Flow Duration Curve.—A curve secured from an arrangement of daily stream flows in the order of their magnitudes. The ordinates represent flow, and the abscissa for any ordinate represents the percentage of time during which flows greater than that ordinate occur.

Output Duration Curve.—A curve secured from an arrangement of daily stream power capacities in the order of their magnitudes. The ordinates

represent kilowatts, and the abscissa for any ordinate represents the percentage of time during which stream-power capacities greater than that ordinate occur.

Reservoir Replenishment, or Rate of Reservoir Filling.—The rate of inflow to the reservoir, minus the rate of draft. Replenishment occurs only when the inflow exceeds the draft, and hence it is always a positive quantity.

Reservoir Depletion, or Rate of Reservoir Emptying.—The rate of draft on the reservoir, minus the rate of inflow. Depletion occurs only when the draft exceeds the inflow, and hence it, too, is always a positive quantity.

Run-of-River Plant.—A hydro-electric plant that depends upon the daily flow of the river for its energy. It may have pondage, but not storage.

Storage Plant.—A hydro-electric plant that receives all its water directly from a large reservoir. The output of a storage plant is not affected by daily fluctuations in flow.

Partial Storage Plant.—A hydro-electric plant the tributary drainage area of which contains a storage reservoir with no plant intervening between it and the plant in question. Part of the flow from the drainage area above the plant is regulated by the storage reservoir. The remainder of the flow is unregulated.

Primary Energy.—Dependable energy from a plant or group of plants; that is, energy that can be relied upon for carrying a portion of the system load for which no other source of supply is regularly maintained.

Secondary Energy.—Energy other than primary, generated by the plants.

Waste Energy.—Energy lost in water that is by-passed necessarily, usually resulting from floods.

Controlled Energy.—Energy obtained from such flow as is regulated by a storage reservoir.

Uncontrolled Energy.—Energy obtained from unregulated flow.

Potential Energy.—Energy at a plant as limited by stream flow, plant efficiency, and average net head.

Basic Assumptions

The duration-curve method, as developed in this study, is based on the following assumptions:

(1) That the streams under consideration have the same distribution of flow; that is, flows that have the same duration-curve abscissa occur at the same time on each of the streams.

(2) That the water discharged from an upper plant is immediately available for use at the next down-stream plant.

(3) That the storage available at each reservoir is unlimited.

General Description

The first step in solving any hydro-electric problem by the duration curve is the selection of a period of flow for analysis, generally from a study of the mass curve. The selected period ordinarily will extend over several dry

years, and will give the minimum reliable rate of controlled flow from storage. Its length, however, may vary from a few months to several years, depending upon the nature of the storage facilities.

It is axiomatic that the duration-curve method gives results for the period for which the curves are made and that the curves must be made up from all, but no more than, the data for the period chosen. In other words, the duration curve for determining dependable output and plant capacities must be prepared from flow records for a complete cycle, a resorvoir cycle being the elapsed time from full reservoir to full reservoir.

The duration curve for determining storage reservoir capacity must be prepared from flow records either for the period from the beginning of the cycle to the time of maximum draw-down, or for the period from the maximum draw-down to the end of the cycle.

The entire drainage area supplying water to the plants of a combined development is divided into a number of parts, each part being the area, tributary to a plant, below all up-stream dams. An output duration curve is then prepared for each of the partial areas. Each of these curves includes all the potential energy derived from flow from the area in question, at all the down-stream plants. For reasons which will appear later, the potential energy of each plant is shown separately on the duration-curve diagram.

After constructing output duration curves for each area, the curves representing uncontrolled energy are added, in order to make a composite curve. This is done by adding graphically the ordinates corresponding to the same abscissa of each of the separate uncontrolled energy curves. The curves of controlled energy are then added to the composite curve of uncontrolled energy in such a way as to give the maximum dependable energy output for the resulting combined uncontrolled and controlled energy curve. The plant capacities, as well as the maximum dependable energy output rate, are read directly from this final curve.

The maximum dependable energy output rate is applied to the storage-requirement duration curves in order to determine the demand on the stored water. The difference between the reservoir draft and inflow is the required storage capacity.

The duration-curve method may best be explained by considering its application to a few hypothetical cases.

Theoretical Applications

Case 1.—Combination of a Run-of-River Plant and a Storage Plant on Adjacent Streams.—It is often desirable to install hydro-electric plants on two adjacent streams in such a manner that the two plants in combination will maintain a uniform dependable output. It may be that only one of the streams presents storage facilities, so that the problem of determining the dependable output becomes complicated. Accordingly, the method of handling the problems presented by two such plants is discussed herein.

It is assumed that in the river basin of Fig 1 there is a run-of-river plant, *B*, and a storage plant, *C*, and that any desired plant capacities can be

installed at each of them. Flow duration curves are not used directly for determining output rate and plant capacity; hence, they need not be constructed. The output duration curves are constructed by collecting and tabulating the daily flows as if to make a flow duration curve. It will be found convenient to collect the daily flows in groups. The range of each group should not exceed 5% of its median value, in order to insure accurate results. If the daily flows are thus arranged, it is important that each group be represented by its lowest value and not by its mean, because any point on

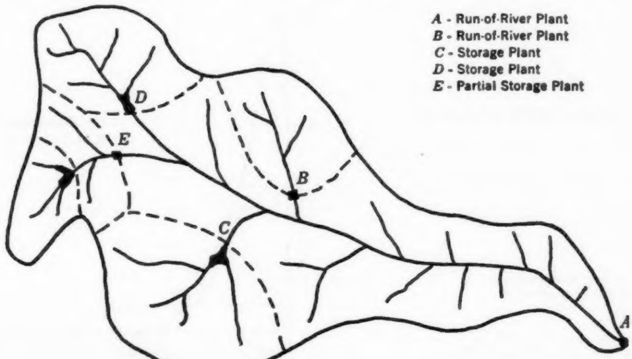

FIG. 1.—RELATIVE LOCATION OF PLANTS

a duration curve indicates the time during which that particular flow is equalled or exceeded, and use of the mean will give duration curves with ordinates that are too large. These daily values (or groups of values) are then multiplied by a constant for each particular plant to convert them to power. This constant is the term, $\frac{H\,e}{11.8}$, in the equation:

$$KW = Q\,\frac{H\,e}{11.8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, KW is output, in kilowatts; Q is flow, in cubic feet per second; H, head on plant, in feet; and, e, plant efficiency. These power ordinates are plotted against the percentage of total time during which they are equalled or exceeded, thus forming the output duration curve.

In this example the area under the curve of Fig. 2(a) represents the total potential energy at Plant C. (The subdivided areas refer to Case 2, explained subsequently.) As drawn, the curve indicates the normal stream frequency, but as the flow from the area above Plant C can be controlled completely by a storage reservoir, it may be converted into energy at such times and at such rates as desired; hence it is not correct to state that Fig. 2(a) represents the output frequency of Plant C. This curve is useful only as an area; consequently, in practice, it is not necessary to construct it but merely to compute its area from the power values of the duration tabulation, which is more accurate than measuring the attenuated area of a duration curve with a planimeter.

The area under Curve *B* in Fig. 2(*b*) represents the total potential energy at Plant *B*, a run-of-river plant with uncontrolled flow that must be used as it occurs.

The determination of the primary energy obtainable from the combination of Plants *B* and *C* is made as follows: It is assumed that the demand for power is of such a nature that a uniform rate of output is required. This is represented on a duration curve by a horizontal line, indicating an equal rate of output at all times. The ordinates of Curve *B*, Fig. 2(*b*), cannot be

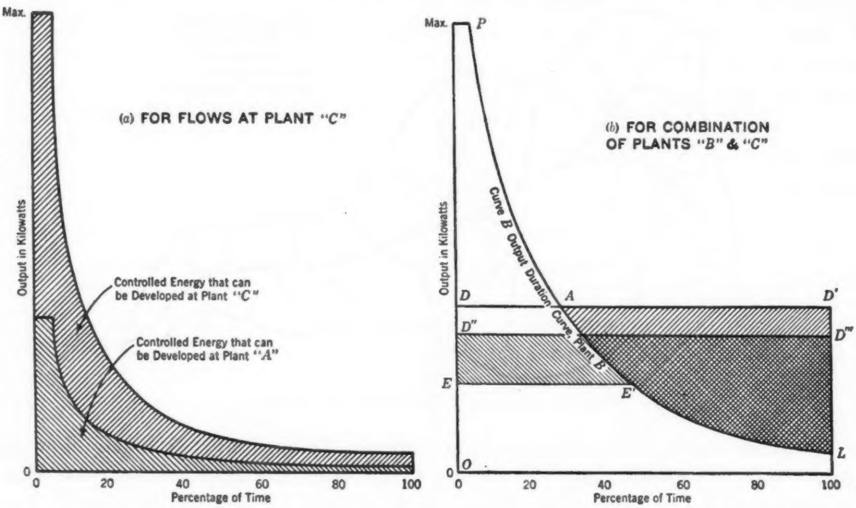

FIG. 2.—OUTPUT DURATION CURVES SHOWING PRIMARY ENERGY OBTAINABLE FROM PLANTS *B* AND *C*.

changed since Plant *B* has no storage and the power must be developed when the water is available, or it will be lost. Since the energy of Plant *C*, represented in Fig. 2(*a*), is all controlled energy, it may be drawn upon in such amounts and at such times as desired. If it is drawn in such a manner as to furnish a uniform rate of energy output from the combination with Plant *B*, the duration curve of their combined energy output may be shown graphically as Line *DD'* in Fig. 2(*b*). The energy output of Plant *C* is represented by the shaded area, *AD'L*, which equals the area of Fig. 2(*a*), and represents the manner in which the output from Plant *C* is drawn. The total area under the line, *DD'*, is the total primary energy that can be developed at the two plants.

The installed capacities required for each of the two plants can be read from Fig. 2(*b*). The ordinate, *OD*, is the dependable output of the two plants, and it also represents the installed capacity required at Plant *B* to maintain the uniform output, *OD*; the ordinate, *LD'*, represents the similar required capacity at Plant *C*.

The area above *DA* and under the curve represents potential energy at Plant *B* that cannot be developed with a plant capacity limited to *OD*. Part

of it may be realized as secondary energy by increasing the capacity of Plant *B*. The amount of such increase, if any, should be determined by a cost and value study.

The next step is the determination of the storage reservoir capacity controlling the flow to Plant *C*. This is accomplished by means of output duration curves for the flow occurring during the period extending from the time of full reservoir to the time of maximum draw-down. The month of maximum draw-down can easily be determined from a mass curve of reservoir inflow plotted by months. The day of maximum draw-down can then be determined by plotting the mass curve of reservoir inflow for the critical month.

Output duration curves for the flow at Plants *B* and *C* for this period (from full reservoir to point of maximum draw-down) are constructed as shown in Fig. 3. The dependable output of the two plants, as represented by the ordinate, *OD*, of Fig. 2(*b*), is applied to the duration curve for Plant *B* (Fig. 3(*a*)) to obtain the horizontal line, *DD′*. The area, *MD′N*, repre-

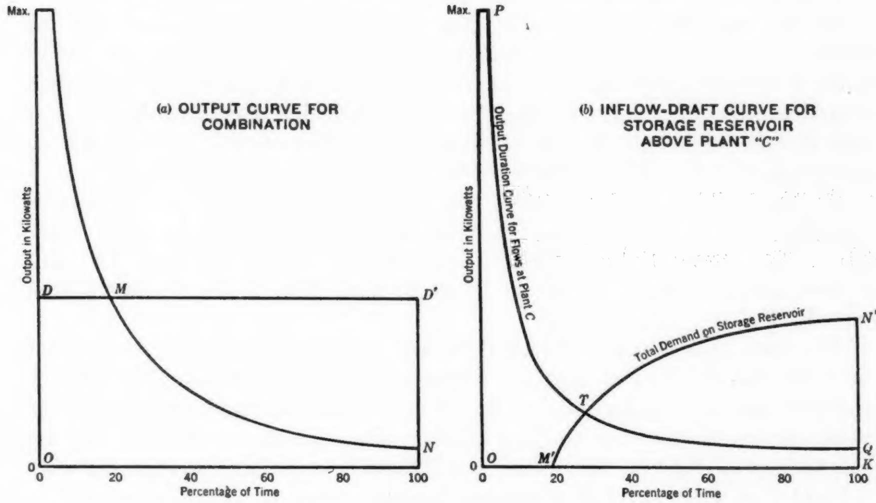

FIG. 3.—DURATION CURVES FOR COMPUTING STORAGE CAPACITY CONTROLLING FLOW OF PLANT *C*.

sents the total demand on the storage reservoir for that period. This demand is plotted in Fig. 3(*b*) with the inflow duration curve for Plant *C* (full reservoir to point of maximum draw-down). The curve, *PTQ*, is the duration curve of inflow to the reservoir in terms of potential energy. The curve, *M′TN′*, is the duration curve of energy demand at Plant *C*; the ordinates are those of the area, *MD′N*, in Fig. 3(*a*).

In Fig. 3(*b*), the area, *M′TQK*, is common to both the inflow and outflow areas and represents energy that is drawn from the reservoir as fast as it enters. The area, *TN′Q*, represents that demand on energy which must be

provided by storage, and the area, $OPTM'$, that energy from inflow which must be stored until needed. The ordinates of the area, $TN'Q$, represent reservoir depletion and those of the area, $OPTM'$, reservoir replenishment.

Since the area, $M'TN'K$, represents the total demand on the storage reservoir and the area, $OPTQK$, the total inflow to the storage reservoir during the period from full reservoir to point of maximum draw-down, the difference between these areas must represent the net amount drawn from the storage reservoir during the period. Since the period extended to the point of maximum draw-down, the net amount drawn from the reservoir represents the maximum storage capacity needed in that period. Of course, the area is in terms of energy and must be transformed to second-foot-days, or acre-feet, by using the energy constant previously determined for Plant *C*.

In a similar manner, the draw-down at any point can be obtained from duration curves constructed for the flow during the period beginning with full reservoir and ending with the point studied. All the duration curves needed for a study can be made at the same time by distributing the data into the necessary periods when the flows are being collected into groups.

In the preceding discussion of plant capacities it was assumed that the desired plant capacity could be installed at *B*. In some cases the capacity of Plant *B* as determined by the ordinate, OD (Fig. 2(*b*)), may be larger than is needed. In other cases, Plant *B* may have been installed already and it is then desired to install the second plant on another stream with a view to regulating the output from both plants.

If Plant *B* has been installed, the method of attacking the problem is a modification of the foregoing. Let the capacity of Plant *B* be some quantity as OE shown in Fig. 2(*b*). Then, the area under the line, EE', can be developed as primary energy at Plant *B*, while that above the line will be wasted.

The area representing controlled energy (Fig. 2(*a*)) now may be placed above the line, $EE'L$ (Fig. 2(*b*)), as indicated by the shaded area, $D''D'''LE'E$. As before, the area under the line, $D''D'''$, represents the total primary energy which can be developed at the two plants. The capacity of Plant *C* may be read from Fig. 2(*b*) as the ordinate, LD'''. The necessary storage capacity at Plant *C* is determined in a manner similar to that previously described.

Thus far, it has been assumed that the reservoir capacity is unlimited. If the storage requirement as determined is too large to be developed economically, the procedure is to return to the mass curve, lower the rate line, and use the data for the shortened period in another study.

It may be noted here that the rate line used on a mass curve of flow, from the area above Plant *B*, say, is not the rate line for the combination of plants; but it may be used to determine the period of reservoir depletion and replenishment when the streams have the same distribution of flow, as stated in the first basic assumption.

Case 2.—Combination of a Run-of-River Plant and a Storage Plant on the Same Stream.—The treatment of the case in which the two plants are on the same stream is very similar to that in Case 1. Fig. 1 shows the respec-

tive locations of the plants, A being a run-of-river plant, and C, a storage plant with complete control of the flow above C. First, consider that unlimited plant capacity can be installed at both plants.

The outstanding feature of this set-up, differentiating it from the first set-up, is the fact that all water used for energy at Plant C is again available at Plant A, where it must either be utilized for energy or by-passed. Determination of the quantity by-passed is the problem demanding solution.

The curve, PSL, of Fig. 4(a) is the output duration curve for flows at Plant A, which, of course, does not include the flows from the area above

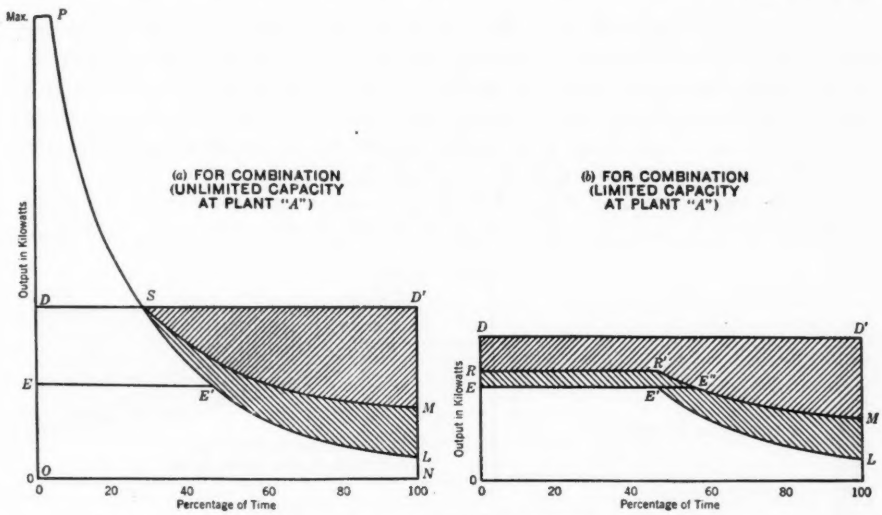

FIG. 4.—OUTPUT DURATION CURVES FOR CASE 2; RUN-OF-RIVER PLANT AND STORAGE PLANT ON SAME STREAM

Plant C. The output duration curve for flows from the area above Plant C as drawn in Fig. 2(a) differentiates between the amounts of potential energy at Plants A and C. For example, 1 cu. ft. per sec. from the area above Plant C is considered to be capable of generating two units of energy at Plant C and one unit of energy at Plant A.

The maximum uniform dependable output from the combination may be obtained as explained for the preceding case. In Fig. 4 (a), it will be noted that each ordinate of the area representing controlled energy is still so divided as to represent two units of energy developed at Plant C for each unit of energy developed at Plant A. The ordinate, OD, represents the uniform dependable output from the combination and also the capacity to be installed at Plant A. The ordinate, MD', represents the capacity to be installed at Plant C. The area, $ODSMN$, represents the energy developed at Plant A, and the area, $SD'M$, the energy developed at Plant C.

When the capacity of Plant A is limited to OE, as in Fig. 4(a), the areas of Fig. 2(a) are applied as shown in Fig. 4(b). Since the capacity of Plant A is limited to OE, all potential energy (Area $EE''R'R$) above the

line, EE'', that is shown as being available at that plant, cannot be developed and must be by-passed. The effect of this reduction in energy is to lower the rate line, DD' until this area is compensated.

In making this adjustment, it should be noted that at all points to the right of E'', the ratio of controlled energy developed at Plant C to controlled energy developed at Plant A must be the same after the adjustment as before. The installed capacity at Plant C is equal to the ordinate, MD', of Fig. 4(*b*) after the adjustment has been made. The reservoir capacity required at Plant C may be determined in a manner similar to that described in Case 1.

Case 3.—Combination of a Run-of-River Plant and a Partial Storage Plant on the Same Stream.—Consider the case of a partial storage plant, E, on the same stream and above A in Fig. 1. Water from the storage reservoir may be carried to Plant E by a canal, tunnel, or other suitable means. Provision is also made at Plant E to utilize the energy of the flow from the area below the storage reservoir and above Plant E.

Let the curve, PSL, of Fig. 4(*a*), be the output duration curve for flows at Plant A (not including those from above Plant E). Fig. 5(*a*) is a com-

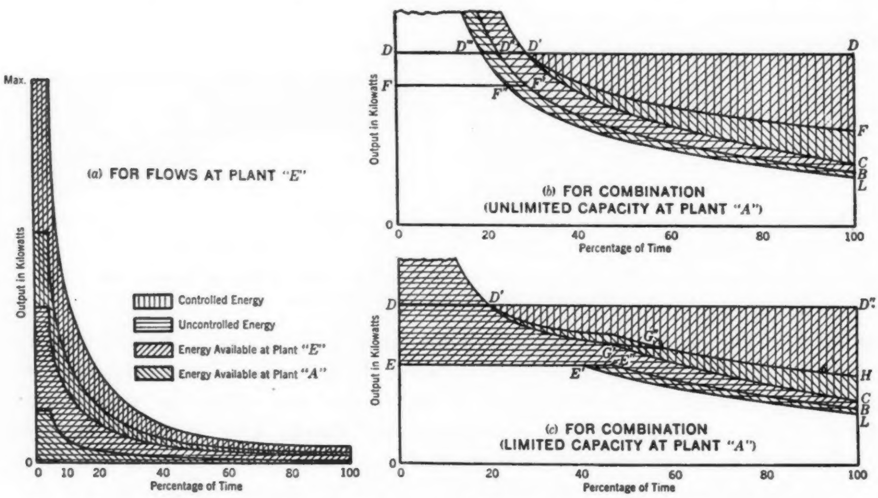

FIG. 5.—OUTPUT DURATION CURVES FOR CASE 3; RUN-OF-RIVER PLANT AND PARTIAL STORAGE PLANT ON SAME STREAM

posite output duration curve for flows at Plant E, distinguishing not only between flows from the area below the reservoir (uncontrolled) and flows at the reservoir dam site (controlled), but also between the potential energy of these flows at Plants E and A.

Fig. 5(*b*) shows the re-arrangement of the duration curves when unlimited capacity is available at both plants. The uncontrolled energy ordinates of Figs. 4(*a*) and 5(*a*) are added graphically and represent the total energy which must either be utilized immediately, or lost. The area representing controlled energy is distributed as shown. The ordinate, OD,

represents the dependable output from the combination of plants. The ordinate, OD, also represents the capacity of Plant A. The sum of the ordinates, BC and FD, represents the capacity of Plant E.

With regard to the production of primary energy: It may be seen by referring to Fig. 5(*b*) that, with the arrangement there shown, Plant A is forced to operate at the capacity, OD, during periods of high flow, while Plant E does not begin producing primary energy until the point, D'', is reached. It will be noticed that potential uncontrolled energy at Plant E in amounts greater than the ordinate, FD, is available at any point to the left of D', indicating that the capacity of Plant A may be reduced to OF and that Plant E may be operated at a rate, FD, until D' is reached, at which point draft on storage begins. It should be noted in this connection that when the capacity of Plant A is reduced to OF, all the potential energy at that plant from the drainage area above Plant E lying above line, FF' (Area $D'''D''F'F''$), cannot be developed and, therefore, is wasted.

The total energy developed at Plants A and E is represented by the area under DD. The division of energy between the plants is indicated by the legend. The storage requirement for the reservoir above Plant E is determined in a manner similar to that described for Case 1.

Thus far, in the discussion of Case 3, the capacity of both plants has been considered as unlimited and determined solely as that capacity necessary to maintain a uniform dependable output. Consider, now, the case in which the capacity of Plant A is limited. Unless the limiting capacity of Plant A is less than OF, the arrangement of the curves does not differ from that of Fig. 5(*b*). When the capacity of Plant A is less than OF, the arrangement is similar, but with a slight modification. Suppose the capacity of Plant A is OE, in Fig. 5(*c*). The uncontrolled energy that can be developed at Plant A from the area between Plants A and E is represented by the unshaded area. To this is added the uncontrolled energy from the area above Plant E represented by the area, $EDG'CLE'$. Part of this is potential energy at Plant E and part at Plant A. At points on the right of E'' all potential energy at Plant A actually may be developed. Between E'' and E', only a part of this energy may be developed at Plant A and on the left of E' none of it may be developed. Accordingly, the only potential uncontrolled energy above EE'' is that available at Plant E. In Fig. 5(*c*), all potential uncontrolled energy not available at Plant A has been omitted above EE''. The potential controlled energy at both plants (Fig. 5(*a*)) is then applied to Fig. 5(*c*) (Area $D'D''CG'$) to maintain a uniform dependable rate, shown by the line, DD''.

None of the potential controlled energy at Plant A can be developed at points to the left of E'' in Fig. 5(*c*), and only a part of such energy for a short distance to the right of E''. The area, $G'D'G''$, representing this wasted energy must be discarded and the vacant space filled with energy available at Plant E. The wasted energy lowers the line, DD'', and reduces the uniform dependable output.

The storage requirement for the reservoir above Plant E may be determined in a manner similar to that outlined for Case 1.

Case 4.—Combination of a Run-of-River Plant and Two Storage Plants.—One more set-up will be discussed briefly to illustrate further possibilities. Consider the case of three plants—*A, C,* and *D*—arranged as illustrated in Fig. 1, with Plant *A*, a run-of-river plant, and Plants *C* and *D*, storage plants located above Plant *A*, but on different branches of the main stream. To facilitate comparison, assume that Plants *C* and *D* have identical possibilities for power development. Fig. 6(*a*) shows one distribution of the controlled

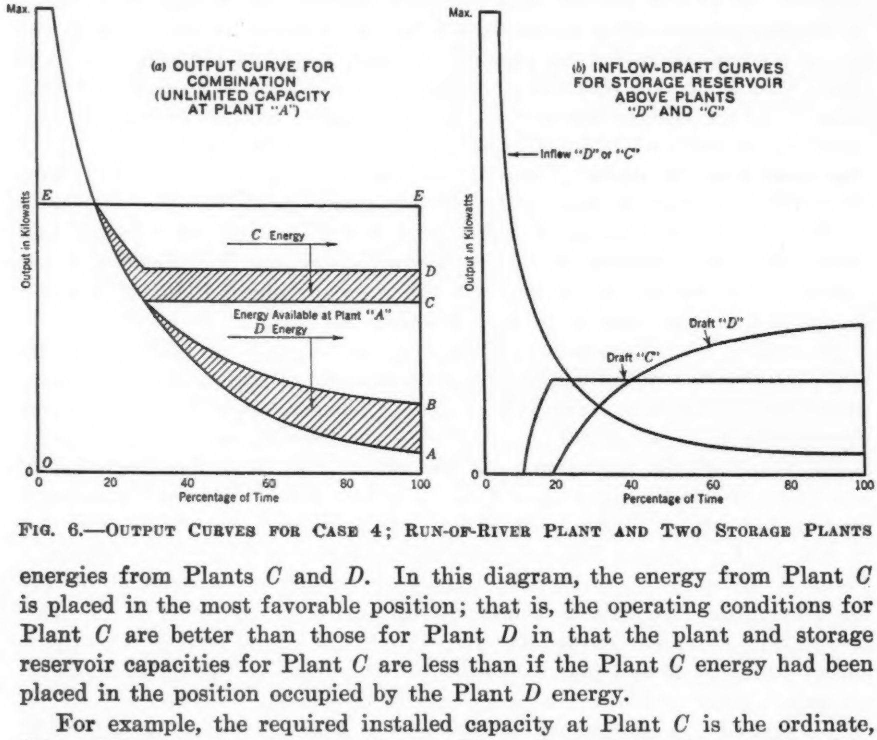

FIG. 6.—OUTPUT CURVES FOR CASE 4; RUN-OF-RIVER PLANT AND TWO STORAGE PLANTS

energies from Plants *C* and *D*. In this diagram, the energy from Plant *C* is placed in the most favorable position; that is, the operating conditions for Plant *C* are better than those for Plant *D* in that the plant and storage reservoir capacities for Plant *C* are less than if the Plant *C* energy had been placed in the position occupied by the Plant *D* energy.

For example, the required installed capacity at Plant *C* is the ordinate, *DE,* while the necessary capacity at Plant *D* is the ordinate, *BC*. The advantage enjoyed by Plant *C* with regard to storage reservoir capacity requirements may be seen by referring to Fig. 6(*b*). While for the complete reservoir cycle the energy produced at Plant *D* equals that produced at Plant *C,* for the period from full reservoir to point of maximum draw-down the energy demand on Plant *D* is greater than that on Plant *C* as shown by the areas under the respective draft curves of Fig. 6(*b*). Since the storage requirement is the difference between the areas under the draft and inflow curves, the storage requirement of Plant *C* will be the smaller.

It is possible to operate Plants *C* and *D* so that their plant and storage capacities are the same. This may be accomplished by distributing the controlled energies so that the outputs of the two plants are equal at all times. Since Plants *C* and *D* usually do not have similar characteristics, the advantage of one over the other may be measured by considering the value of the

output weighed against the cost of development, as indicated by the required plant and reservoir capacities. If capacity at Plant A is limited, the procedure is the same, except that care must be taken to discard all potential energy at Plant A that cannot be developed when that plant is operating at its maximum capacity. Storage requirements are handled as in all other cases.

It is thought that sufficient examples have been given to illustrate the method of using duration curves. The cases selected were purposely stripped of all unnecessary complications in order that the basic ideas might not be obscured. Consider a single example of eliminated detail: In any practical case of a combination of plants, the question of load factor becomes very important. If the hydro-electric plant is being operated with steam plants and is used only to carry the peak load, the required installed capacity may be several times that indicated by the duration curve ordinate.

Accuracy

The duration curve method has been tested for accuracy by comparing it with the hydrograph method, using the same base data. The comparison was made for two proposed plant combinations in the Cheat River Basin, namely, (*a*) the Big Sandy development in combination with the Lake Lynn development (Case 2); and (*b*), a Blackwater two-plant development in combination with the Lake Lynn development (Case 3, extended). Furthermore, the comparison has been made (*c*) for a hypothetical partial storage Blackwater development in combination with the Lake Lynn development (Case 3).

The Big Sandy and Blackwater plants are planned to operate in combination with the Lake Lynn hydro-electric plant and large base load steam plants. Accordingly, in Studies (*a*) and (*b*) it was assumed that the hydro-electric plants would be operated so as to carry the peak of the system load. Therefore, plant and system load factors were considered. In Study (*c*), a load factor of 100% was assumed for both plants.

The detailed investigation involved studies of the reservoir cycles by the hydrograph method, using weekly flows. In addition, the daily flows were scanned, and all large flows were properly divided into primary, secondary, and waste energy, so that, for all practical purposes, the investigations were studies of daily flows.

The reservoir storage capacity was determined in Studies (*a*) and (*b*) by a modification of the method described in this paper, in which accuracy was sacrificed for speed. Study (*c*) was made to check the exact method of determining reservoir storage capacity and, hence, was stripped of unnecessary complications. In these studies, the desirable plant capacities were determined in a manner common to both the hydrograph and duration curve methods.

Table 1 records the percentage variation obtained by the two methods for the various quantities involved. In evaluating the "error," the hydrograph method was considered correct and the duration curve method at fault.

It may be seen that the error in the determination of the total primary energy was less than 0.5 per cent. For Studies (*a*) and (*b*), in which a

modified method of storage determination was used, the storage capacity required was obtained within an error of 1.5 per cent. On the other hand, when the exact method was used, as in Study (*c*), the error was reduced to less than 0.5 per cent.

TABLE 1.—COMPARISON OF RESULTS

Item No.	Description*	Hydrograph method	Duration curve method	Error	Error (percentages)
	STUDY (*a*) — BIG SANDY DEVELOPMENT				
1	Lake Lynn uncontrolled Class *A* primary energy	503 400	491 000	—12 400	— 2.46
2	Lake Lynn controlled Class *A* primary energy	87 000	94 500	+ 7 500	+ 0.86
3	Big Sandy controlled Class *A* primary energy	760 900	772 400	+11 500	+ 1.51
4	Total Class *A* primary energy	1 351 300	1 357 900	+ 6 600	+ 0.49
5	Class *B* primary energy	89 000	89 700	+ 700	+ 0.79
6	Secondary energy	158 200	154 200	— 4 000	— 2.53
7	Waste energy	110 500	104 300	— 6 200	— 5.61
8	Plant use	5 100	4 900	— 200	— 3.92
9	Total energy	1 714 100	1 711 000	— 3 100	— 0.18
10	Reservoir capacity, in thousands of acre-feet	299	298	— 1	— 0.33
	STUDY (*b*) — BLACKWATER DEVELOPMENT				
11	Lake Lynn uncontrolled Class *A* primary energy	528 900	511 800	—17 100	— 3.23
12	Lake Lynn controlled Class *A* primary energy	41 500	49 500	+ 8 000	+19.28
13	Blackwater uncontrolled Class *A* primary energy	294 700	299 900	+ 5 200	+ 1.76
14	Blackwater controlled Class *A* primary energy	560 100	564 400	+ 4 300	+ 0.77
15	Total Class *A* primary energy	1 425 200	1 425 600	+ 400	+ 0.03
16	Class *B* primary energy	94 400	94 200	— 200	— 0.21
17	Secondary energy	186 500	188 500	+ 2 000	+ 1.07
18	Waste energy	119 700	141 300	+21 600	+18.05
19	Plant use	5 800	4 600	— 1 200	—20.69
20	Total energy	1 831 600	1 854 200	+22 600	+ 1.23
21	Reservoir capacity, in thousands of acre-feet	132.7	131.0	— 1.7	— 1.28
22	Secondary and waste energy at Blackwater	14 300	8 600	— 5 700	—39.9
	STUDY (*c*) — BLACKWATER DEVELOPMENT				
23	Lake Lynn uncontrolled primary energy	112 670	112 700	+ 30	+ 0.03
24	Lake Lynn controlled primary energy	8 208	8 111	— 97	— 1.18
25	Blackwater uncontrolled primary energy	34 707	33 460	— 1 247	— 3.59
26	Blackwater controlled primary energy	113 463	113 610	+ 147	+ 0.13
27	Total primary energy	269 048	267 881	— 1 167	— 0.43
28	Secondary energy	22 126	24 364	+ 2 238	+10.11
29	Waste energy	62 122	61 119	— 1 003	— 1.61
30	Total energy	353 457	353 364	— 93	— 0.03
31	Lake Lynn plant, capacity, in kilowatts	25 000	25 000	......	......
32	Blackwater plant, capacity, in kilowatts	28 000	28 400	— 200	— 0.70
33	Reservoir capacity, in thousands of acre-feet	110.7	110.9	+ 0.2	+ 0.18

* Unless otherwise noted all values, multiplied by 1 000, are in kilowatt-hours.

The error involved in the use of the duration curve method is dependent upon the degree of similarity in flow distribution at the various plants. In the case of Cheat River Basin, there is some dissimilarity in flow among tributaries and the main river. However, the error introduced by the use of the duration curve is not greater than the assumptions as to average head, efficiency, etc.

General Discussion

Applicability.—The duration curve method was developed primarily to expedite preliminary investigations of power and storage projects for conditions in which the streams do not vary widely in their characteristics. Some of the tributaries of the Cheat River are more "flashy" than the main stream, but even so, their hydrographs are strikingly similar in point of flow distribution to those of the main stream.

It is thought that the method is particularly applicable to hydro-electric studies for streams with similar flow characteristics. It can be utilized for preliminary comparative studies of projects, for studies of developments involving two or more storage reservoirs, and for storage studies in which ample storage is available at a reasonable cost. A shorter cycle of reservoir depletion and replenishment must be used in cases where ample storage is not available at a reasonable cost.

In the theoretical discussion, it was assumed that the water discharged from an upper plant would be immediately available for use at the next down-stream plant. Such a condition could never exist, but, in practice, the manner of plant operation will eliminate much of the error caused by this assumption. Most plants are supplied from storage or diversion ponds which are able to store the inflow of one or more days. Water in quantities equivalent to that discharged at an up-stream plant can be drawn from the pond with the certainty that it will be refilled by the up-stream flow. In general, such operation will enable the duration curve method to be used for many hydro-electric studies.

Many factors have been entirely omitted from the discussion in order that the process might not be encumbered with too much detail. Among the more important are load factor (mentioned previously), seasonal variation of load, regulation of flow by diversion dams, variation of head due to reservoir draw-down, evaporation from the reservoir, etc.

The duration curve method should be used with caution, or not at all, where streams on widely separated water-sheds are being considered for obtaining a uniform power rate from two or more hydro-electric developments.

Advantages.—The method outlined has several advantages that recommend it for preliminary investigation. It is rapid and accurate, requiring little work for preparation, except the construction of flow duration curves for the plants considered. It eliminates much of the "cut-and-try" inherent in the week-by-week hydrograph studies wherein a rate of use must be assumed and a study completed with this rate before it is known whether the reservoir will fill. The duration curve method insures a full reservoir at the end of the period of study. The results, when finally obtained, are presented graphically, showing more readily and effectively than any tabulation, the distribution of load and the effects to be expected from a change in that distribution.

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REPORTS

STANDARD SYMBOLS AND GLOSSARY FOR HYDRAULICS AND IRRIGATION

COMPILED BY

THE SPECIAL COMMITTEE ON IRRIGATION HYDRAULICS [1]

FOREWORD

Among engineers engaged in hydraulic and irrigation problems, a difference of opinion has long existed regarding the definition and interpretation of certain commonly used terms. The symbols and glossary submitted herewith constitutes one attempt to answer the perennial questions raised by these variations of interpretation.

Realizing the need for greater uniformity of nomenclature, the Society's Special Committee on Irrigation Hydraulics determined that the best way of solving the questions was to set up a proposed series of standard definitions which, after discussion, might be construed as having the best obtainable measure of authority. Accordingly, the Committee authorized a Sub-Committee to study and formulate its best thoughts on this matter. About four years of work on the part of this Sub-Committee is presented herewith. The very fact that differences of opinion were pronounced and that the definitions here recommended were often the result of much discussion, indicates the necessity for standard definitions.

In order that these forms may be brought to the attention of every member, the Society is printing the complete list in *Proceedings*. It may then be studied at leisure; may be used in daily work by members, and thus tested by experience; and, finally, any difficulties or corrections may be noted and submitted to the Society. Such comments are welcome; they will be brought to the attention of the Committee for its use in possible revision of the standard symbols and glossary.

For the Special Committee on Irrigation Hydraulics,

J. C. STEVENS,

Secretary.

March 16, 1932.

[1] The members of the Special Committee on Irrigation Hydraulics are, as follows: Messrs. D. C. Henny, *Chairman*, J. C. Stevens, *Secretary*, B. A. Etcheverry, George W. Hawley, Julian Hinds, Ivan E. Houk, R. L. Parshall, J. L. Savage, Fred C. Scobey, I. C. Steele, and Franklin Thomas.

STANDARD SYMBOLS FOR IRRIGATION AND HYDRAULICS[2]

A = Major hydraulic area, as conduit cross-section; cross-sectional area of a water prism.

a = (1) Minor, partial, or local hydraulic areas.
$a_1, a_2, \ldots$ = areas at particular sections.
(2) Acceleration.

b = Width of bed of channel.
$b_1, b_2, \ldots$ = widths at particular sections.

C = (1) Chezy coefficient in formula, $V = C \sqrt{R f}$.
(2) Weir coefficient.
(3) Special coefficients.

c = Coefficient or constant.
c_c = coefficient of contraction.
c_v = velocity.
c_q = flow, etc.

D = Diameter; unless expressed otherwise it will refer to inside diameter.

d = (1) Diameter.
(2) Differential.
(3) Depth of water in channel.
$d_1, d_2, \ldots$ = depths at particular sections.
d_c = critical depth.
d_n = neutral depth.
See, also, y.

E = (1) Total energy.
(2) Modulus of elasticity.

e = (1) Efficiency.
(2) Energy head.
$e_1, e_2, \ldots$ = energy heads at particular sections.
(3) Base of Naperian logarithms.

F = (1) Force.
(2) Function of . . .

f = (1) Hydraulic friction factor in Weisbach's formula.
(2) Sine of slope due to hydraulic friction in open channels.
(3) Function of . . .

g = Acceleration of gravity, 32.2 ft. (or 9.80 m.) per sec. per sec., for ordinary computations.

H = (1) Major head, from head-water to tail-water; other total heads.
(2) Loss of head per 1 000 ft. of pipe, in exponential formulas.
(3) A horizontal force.

h = (1) Minor head.
h_v = velocity head.
h_f = friction head.

[2] Where no confusion will result the subscripts may be used alone for the sake of brevity, thus, i, eddy loss; e, energy head; h, velocity head; u, uplift pressure; etc.

h_e = energy head.
h_i = head lost by impact and eddies.
h_p = pressure head.

(2) Velocity head.
$h_1, h_2, \ldots$ = velocity heads at particular sections.
h_c = velocity head of critical velocity.
h_a = velocity head of approaching water.

(3) Height of structure.
(4) Height of water on a gage; a gage reading.

I = Moment of inertia; impulse of a force.
i = Head lost by impact and eddies.
J = Height of hydraulic jump expressed as a ratio of depth after, to that before, the jump.
K = Constants, usually with subscript.
k = (1) Elevation of stream bed above datum.
$k_1, k_2, \ldots$ = elevations at particular sections.
(2) Constants, with subscripts.
(3) Radius of gyration.

L = (1) Length of conduit; that is, developed length.
(2) Length of weir crest.

M = (1) Major mass.
(2) Momentum.

m = (1) Minor mass.
(2) Coefficient of hydraulic friction in Bazin's formula.

N = (1) Coefficient in formula for submerged weirs, to express degree of submergence.
(2) Number of contractions in weir flows.

n = (1) Coefficient of hydraulic friction in Kutter's and Manning's formulas, distinguished thus; Kutter's n and Manning's n.
(2) An exponent.
(3) A subscript to denote depth.
(4) Number of things.

O = Origin of co-ordinates.
P = (1) Wet perimeter.
(2) Total pressure.
P_n = Normal pressure.
P_r = Resultant pressure.
P_h = Horizontal component of pressure.
P_v = Vertical component.
P_a = Atmospheric pressure.
P_u = Uplift pressure.

p = (1) Local or partial wet perimeter.
$p_1, p_2, \ldots$ = perimeters at particular sections.
(2) Intensity of pressure.

Q = Major flow; that is, volume per unit of time. (Discharge is much used, but inadequate.)

q = Minor, local, or partial flow.

$q_1, q_2, \ldots$ = flows at particular sections.

q_c = critical flow.

q_n = neutral flow.

R = Major hydraulic radius; that is, ratio of wet area to wet perimeter.

r = Minor hydraulic radius.

$r_1, r_2, \ldots$ = radii at particular sections.

S = Sine of slope of channel bed.

See, also, f.

s = Side slopes in cuts and fills, that is, s, horizontal distance in unit vertical distance, as 1.5:1.

T = (1) Major time.

(2) Summation of t.

(3) Temperature.

(4) Major top width as an alternate to W.

(5) Thickness.

t = (1) Minor time.

(2) Temperature.

(3) Minor top width as an alternate to w.

(4) Minor thickness.

u = Velocity of a particle, or filament.

V = Major mean velocity; that is, flow divided by cross-sectional area of water prism, either at a particular section or throughout a given length of channel.

V_a = velocity of approach.

V_o = Kennedy's critical velocity.

V_c = Unwin's critical velocity.

$V_{max.}$ = maximum velocity, etc.

v = Minor, local, or partial velocity.

$v_1, v_2, \ldots$ = velocities at particular sections.

W = (1) Work.

(2) Weight of given volume.

(3) Major width. (T may be used as an alternate.)

w = (1) Unit weight. (In all ordinary calculations use weight of water as 62.5 lb, per cu. ft.)

(2) Minor width. (t may be used as an alternate.)

$w_1, w_2, \ldots$ = widths at particular sections.

X = Co-ordinate axis.

x = (1) Distance along X-axis.

(2) Exponent.

(3) An unknown quantity.

Y = Co-ordinate axis.

y = (1) Vertical depth.

$y_1, y_2, \ldots$ = depths at particular sections.

(2) Distance along Y-axis.
See, also, d.

Z = Co-ordinate axis.

z = (1) Elevation of water surface above datum.
$z_1, z_2, \ldots$ = elevations at particular points.
(2) Depths normal to channel bed on steep slopes.
(3) Distance along Z-axis.

Glossary of Terms Used in Irrigation and Hydraulics

Absorption Loss.—Loss of water from a canal or reservoir by capillary action and percolation during the process of priming. After a canal or reservoir has reached a stable condition this loss is called "seepage."

Acre-Foot.—Quantity of water that would cover 1 acre, 1 ft. deep. An acre-foot contains 43 560 cu. ft. A second-foot flowing for 24 hours is nearly equivalent to 2 acre-ft., and 1 acre-ft. falling 1 ft. is the approximate energy equivalent of 1 kw-hr.

Aeration.—Relieving the effects of cavitation by admitting air to the section or area affected.

Air-Lift.—A means of raising water by air pressure or by the buoyancy of injected air.

Air Stand.—An air vent on a pipe line.

Alkali.—Soluble salts in soil. In excessive quantities they become injurious to plants.

Anchor-Ice.—Ice forming on the bed of streams in cold, clear weather.

Apron.—An adjunct to a dam or other structure, consisting of a surface protection against erosion.

Aqueduct.—A major conduit.

Arch Dam.—A curved dam convex up stream, that depends on arch action for its stability. The load is transferred by the arch to the canyon walls, or other abutments.

Arid.—A term applied to lands, or climates, that lack sufficient water for agriculture without irrigation.

Automatic Gate.—A gate that operates through water pressure.

Average Velocity.—The arithmetical mean of velocities.

Back-Water Curve.—A particular form of surface curve which is concave upward.
See, also, ***Surface Curve.***

Barrage.—A dam.

Basin Irrigation.—A method of orchard irrigation by which each tree is surrounded by a border.

Bear-Trap Dam.—A dam of hinged leaves, that is raised and held up by the pressure of water admitted to the inside. It is lowered by draining the interior.

Bed Load.—The silt, sand, gravel, or other débris rolled along the bed of a stream.

Bench-Flume.—A flume built on a bench, cut on sloping ground.

Bernoulli's Theorem.—A theorem advanced by Daniel Bernoulli that the energy head at any section in a flowing stream is equal to the sum of that at any other down-stream section and the intervening losses.

Bifurcation Gate.—A structure that divides the flow between two conduits.

Blow-Off.—A controlled outlet on a pipe line to discharge water or detritus.

Border.—An earth ridge thrown up to hold irrigation water within prescribed limits in a field.

Border Irrigation.—An open field method of flood irrigation between controlling ridges or borders.

Bottom Contraction.—(1) The contraction in the area of overflowing water caused by the crest of a weir; (2) contraction of the bottom of the nappe.

Broad-Crested Weirs.—Weirs whose crests have an appreciable stream-wise dimension in contradistinction to sharp-crested weirs.

Broad Irrigation.—Irrigation with sewage, in which sewage disposal is the primary object.

Bucket.—(1) A curved surface at the toe of an overflow dam to deflect the water horizontally; (2) the transition curve between the down-stream face and the apron of a dam.

Caisson.—A closed chamber, usually sunk by excavating within it, for the purpose of sub-aqueous work.

Canal.—An open conduit for the conveyance of water; distinguished from a ditch or lateral by its larger size; usually excavated in natural ground.

Capillary Water.—Water held above the water-table in soil by capillary forces.

Catchment Area.—Water-shed; drainage basin.

Catenary.
See *Hydrostatic Catenary.*

Cavitation.—A condition of partial vacuum produced by flowing water.

Channel.—An elongated open depression in which water may or does flow.

Check.—(1) A structure to raise the water surface in a canal or ditch; (2) in irrigation fields, an area of land enclosed in dikes to confine the irrigation water (a long rectangular area between borders, having a definite slope, is a "strip" and not a "check").

Check Irrigation.—A method by which a field, divided into compartments or checks, is irrigated by pooling water into them successively.

Chemical Gaging (Chemi-Hydrometry).—A process of measuring the flow of water by ascertaining the resulting amount of dilution of a chemical solution of known saturation introduced into the stream at a known rate.

Chute.—(1) A high-velocity conduit for conveying water to a lower level; (2) an inclined drop or fall.

Cipolletti Weir.—A contracted measuring weir, the sides of the notch of which have a slope of 1 horizontal to 4 vertical, to compensate for end contractions; named after Césare Cipolletti, an Italian engineer.

Coefficient of Roughness.—A factor in the Kutter, Manning, Bazin, and other formulas expressing the character of a channel as affecting the friction slope of water flowing therein.

Coffer-Dam.—A dam built in the water, forming an enclosure which, when pumped out, will permit sub-aqueous construction.

Conduit.—A general term including canals, ditches, flumes, pipes, or any other means or devices for the conveyance of water.

Consumptive Use.—Water used by transpiration and evaporation in the production of crops.

Continuous-Flow Irrigation.—A system by which each irrigator receives his allotted quantity of water at a continuous rate.

Continuous Stave-Pipe.—A wooden pipe built in place, consisting of staves held by encircling bands.

Contour Checks.—Compartments of a field made by borders following the contours.

Contracted Weir.—A measuring weir whose crest and sides produce a contraction in the area of the overflowing water.

Contraction.—The decreased cross-sectional area of a jet or nappe after passing an orifice, weir, or notch.
See, also, *Vena Contracta.*

Control.—A cross-section of a conduit where conditions exist that determine the regimen of flow up stream independent of the conditions down stream.

Conversion.—A short conduit for uniting two others having different hydraulic elements; a transition.

Conveyance Loss.—Loss of water from a conduit, due to seepage and evaporation.

Core-Wall.—A wall of masonry, sheet-piling, or puddled clay built inside a dam or embankment to reduce percolation.

Cradle.—A footing structure shaped to fit the conduit it supports.

Crest.—The top of a dam, dike, spillway, or weir.

Crib Dam.—A dam made of timber, forming bays or cells which are filled with stone.

Critical Depth.—The depth of water in a channel corresponding to critical velocity. A given quantity of water in an open conduit may flow at two depths having the same energy head. When these depths coincide, the energy head is a minimum and the corresponding depth is Unwin's critical depth.

Critical Flow.—A condition of flow for which the mean velocity is at one of the critical values.
See, also, *Critical Velocity.*

Critical Velocity.—(1) Reynold's critical velocity is that at which flow changes from stream-line to sinuous or turbulent and where friction ceases to be proportional to the first power of the velocity and becomes proportional to a higher power of the velocity; (2) Kennedy's critical velocity is that in open channels, which will neither pick up nor deposit silt; (3) Unwin's critical velocity is that in open conduits for which the velocity head equals one-half the mean depth, and for which the energy head is a minimum.

Current.—The down-stream moving portion of flowing water.

Current Meter.—A device for determining the velocity of flowing water by ascertaining the speed of rotation of a vane or wheel turned by the water.

Cusec.—A cubic foot per second.

Cut-and-Fill.—A process of building canals by excavating part of the depth and using the excavated material for the adjacent embankments. In a balanced cut-and-fill the excavated material just makes the embankments with an allowance for settlement.

Cut-Off.—A wall, collar, or other structure to reduce percolation of water along otherwise smooth surfaces, or through porous strata.

Cut-Off Trench.—A trench in the base of a dam or other structure filled with relatively impervious material to reduce percolation.

Dam.—A barrier to confine or raise water.

Datum.—Plane of reference for elevations.

Débris.—The sand, soil, and gravel moved by a flowing stream; detritus.

Débris Cone.—A fan-shaped deposit of soil, sand, gravel, and cobbles built up at the point where a mountain stream meets a valley, or otherwise where its velocity is reduced sufficiently to cause such deposits.

Delivery Box.—A structure for the control and measurement of water delivered to a farm unit.

Dental.—A tooth-like projection on an apron, or other surface, to deflect or break the force of flowing water; a form of baffle.

Dentated Sill.—A notched sill at the end of an apron to break the force of flowing water and thus prevent erosion below the apron.

Dike.—An embankment to confine or control water, especially one built along the banks of a river to prevent overflow of low lands; a levee.

Discharge.—Flow; this latter term is recommended as preferable.

Discharge Curve.—A rating curve; a preferable term is "Flow Curve".

Discharge Measurement.—A gaging; the term, "Flow Measurement," is recommended as preferable.

Distributaries.—The smaller conduits taking water out of laterals for delivery to the farms.

Distribution System.—The system of main canal laterals, distributaries, and their appurtenances, conveying irrigation water from the main to the farm units.

Ditch.—An artificial channel distinguished from a canal by its smaller size.

Diversion Dam.—A dam built for the purpose of diverting part or all the water from a stream into a different course.

Diversion Canal.—The canal of an irrigation system, from the point of diversion to the beginning of the distribution system.

Division Box.—A structure for dividing and diverting water into other channels. It may divide all flow *pro rata,* or it may divert a definite quantity regardless of the total flow, within a reasonable tolerance.

Division Gate.—A structure that divides the flow between two or more laterals.

Draft-Tube.—A tube connecting a pump or turbine with water at a lower level.

Drain.—A conduit for carrying off surplus ground or surface water. Closed drains are usually buried.

Drainage.—The process of removing surplus ground or surface water by artificial means.

Drainage District.—An organization of land owners operating under legal regulations for financing, constructing, and operating a drainage system.

Drain-Tile.—Pipe of burned clay, concrete, etc., in short lengths, usually laid with open joints to collect and remove drainage water.

Drop.—(1) A structure for dropping the surface of water in a conduit to a lower level and dissipating its surplus energy. A drop may be vertical or inclined; the latter is called a "chute." (2) A fall.

Drop-Down Curve.—A particular form of surface curve which is convex upward.
See, also *Surface Curve.*

Drum-Gate.—A gate in the form of a sector of a circle hinged at the apex. The arc face effects a water seal with the edge of a recess into which the gate may be lowered. The gate is raised and held up by the pressure of water admitted to the recess from the head-water. It is lowered by closing the inlet port to the recess and draining the water from it.

Duty of Water.—The quantity of irrigation water used under different standards of practice. It will vary from a large use under crude practice to small use approaching consumptive use under good practice. It is simply the measure of the use of water and may be distinguished as head-gate or gross duty, lateral duty, duty at the farms, or net duty, and crop duty for different crops. It may be expressed in depth of water on the land, as a rate of flow per acre for a given time, or as the area per unit of flow for a given time.
See, also, *Consumptive Use: Water Requirement.*

Earth Dam.—A dam composed of earth, clay, sand, or sand and gravel.

Eddy Loss.—The energy lost (converted into heat) by swirls and eddies.

Elastica.
See *Hydrostatic Catenary.*

End Contractions.—The contraction in the area of overflowing water caused by the ends of a weir notch.

Energy.—Kinetic energy is that due to motion, and potential energy is that due to position. In a stream the total energy at any section is represented by the sum of its potential and kinetic energies.
See, also, *Energy Head: Energy Gradient.*

Energy Gradient.—The slope of a line joining the elevations of the energy head of a stream.

Energy Head.—The elevation on the hydraulic gradient at any section plus the velocity head.

Entrance Head.—The head required to cause flow into a conduit. Sometimes used as an equivalent of entrance loss.

Entrance Loss.—The head lost in eddies and friction at the inlet to a conduit or structure.

Escape.—A wasteway in a canal.

Evaporation.—The process by which water passes from liquid or solid state to vapor.

Evapo-Transpiration.—Combined loss of water from soils by evaporation and plant transpiration.

Farm Duty.—Irrigation water delivered to a farm unit.

Fines.—(1) The finer grained particles of a mass of soil, sand, or gravel; (2) in hydraulic sluicing the material that slowly settles to the bottom of a mass of water.

Fish Screen.—A device intended to prevent the entrance of fish into a conduit.

Fishway, Fish Ladder.—An arrangement of drops and pools to render possible the migration of fish around dams or other obstructions in streams.

Flash-Board.—A plank held horizontally in an opening by end guides to control the up-stream water level.

Float Gaging.—Measuring the flow of water by floats.

Flow.—Rate of movement of water in a conduit in terms of volume per time; discharge. The English unit of flow is the cubic foot per second, sometimes shortened to cusec, or second-foot; the metric unit is the liter or cubic meter second.

Flow Measurement.—A gaging.

Flume.—An open conduit of wood, concrete, or metal, on a prepared grade, trestle, or bridge. A flume holds water as a complete structure. A concrete-lined canal would still be a canal without the lining, but the lining supported independently would be a flume.

Forebay.—A reservoir at the head of a penstock.

Foundation Mattress.—A slab of concrete, usually reinforced, placed over a yielding foundation to distribute the superimposed load.

Framed Dam.—A dam, generally built of timber framed to form a water face, supported by struts.

Frazil Ice.—Ice which forms in turbulent water when the water falls to freezing temperature. Named from a French word meaning "cinders," from its resemblance to a clot of floating cinders.

Free-Board.—The distance between the operating depth and the top of the sides of an open conduit or the crest of a dam, left to allow for wave action, floating débris, or any other condition or emergency, without overtopping the structure.

Free Flow.—A condition of flow through or over a structure not affected by submergence.

Free Water.—Water in soil in excess of hydroscopic and capillary water; also, called gravity water.

Free Weir.—A weir that is not submerged; that is, one whose tail-water is below the crest or whose flow is in nowise affected by the elevation of the tail-water.

Frequency Curve.—A graphical representation of the time distribution of an orderly arrangement of magnitudes, such as run-off, precipitation, etc.; a duration curve.

Friction Head.—(Friction loss) the head or energy lost as the result of the contact between a moving prism of water and its containing conduit.

Friction Slope.—The friction head or loss per unit of length of conduit.

Furrow Irrigation.—A method of irrigating by small ditches or furrows leading from a header or supply ditch.

Gage.—(1) A staff graduated to indicate the elevation of a water surface; (2) a device for registering elevations, flow, velocity, pressure, etc. Preferred to "gauge."

Gage Height.—The elevation of a water surface above a datum corresponding to the zero of the staff or other type of gage by which the height is indicated.

Gaging.—A single measurement of flow corresponding to a certain stage.

Gaging Station.—A selective point on a stream equipped with a gage and means of measuring the flow of water.

Gallery.—(1) A sub-surface collecting basin for percolating water; (2) a passageway, as in a dam; (3) a subterranean reservoir.

Giant.—A nozzle, mechanically or hand-controlled, for directing a jet of water for hydraulicking.

Grade.—(1) The slope of a road, channel, or natural ground; (2) the finished surface of a canal, roadbed, embankment or excavation; (3) any surface prepared for the support of a conduit, paving, ties, rails, etc.

Gradient.—Change of elevation per unit length; slope.

Gravity Dam.—A dam depending solely on its weight to resist the water load.

Gravity Water.—(1) Water that moves downward through soil under the influence of gravity, also denoted as free water; (2) a gravity supply of water as distinguished from a pumped supply.

Gross Duty of Water.—The irrigation water diverted at the intake of a canal system, per acre irrigated.

Ground-Water.—Mobile water in soil; free water.

Head.—The height of water above any point or plane of reference.

Head-Gate.—The control works, or the gate itself, at the entrance to a conduit.

Heading.—The complete works at the upper end of a main canal, including diversion dam, head-gates, spillways, etc.; intake; head-works.

Head-Race.—A channel leading water to a water-wheel; a forebay.

Head-Water.—The water just up stream from a structure.

Head-Works.—The diversion structures at the head of a canal; intake.

Hollow Dam.—A dam usually of reinforced concrete consisting essentially of slabs supported by transverse buttresses. The load is taken by the slabs and transferred to the foundations through the buttresses.

Hydraulic Elements.—The depth, area, perimeter, mean depth, hydraulic radius, velocity, energy, and other quantities pertaining to a particular stage of flowing water.

Hydraulic-Fill Dam.—An earth dam in which the central portion is of fines sluiced into place by water.

Hydraulic Grade Line.—In a closed conduit a line joining the elevations to which water could stand in risers. In an open conduit, the hydraulic grade line is the water surface.

Hydraulic Gradient.—The slope of the hydraulic grade line. The slope of the surface of water flowing in an open conduit.

Hydraulic Jump.—The sudden and usually turbulent passage of water from low stage below critical depth to high stage above critical depth.

Hydraulic Radius.—The cross-sectional wet area of a conduit divided by its wet perimeter.

Hydraulic Ram.—A device for lifting water by the water-hamer produced by periodical stoppage of flow.

Hydraulicking.—The process of moving materials by water; hydraulic sluicing.

Hydrograph.—A graph showing the stage, flow, velocity, or other property of water, with respect to time.

Hydrographer.—One in charge of flow measurements or their analyses.

Hydrography.—The science of measuring and analyzing the flow of water.

Hydrostatic Catenary.—The curve assumed by a non-extensible but flexible cord, when subject to a normal load at all points proportional to the distance below the horizontal line joining its supports; also called the "Elastica."

Hygroscopic Coefficient.—The soil moisture, in percentage of dry weight, that a dry soil will absorb in saturated air.

Hygroscopic Moisture.—Immobile soil moisture that can only be driven off by heat.

Impact.—The striking together of two masses. When particles or streams of water suffer impact, energy losses result.

Impact Loss.—The head lost as a result of the impact of particles of water; included in and hardly distinguishable from eddy loss.

Impervious.—That property of a material that prevents percolation.

Improved Venturi Flumes.
See *Parshall Measuring Flume.*

Impulse.—The product of a force and the time during which it acts.

Inclined Gage.—A staff gage on a slope graduated to read vertical heights above the datum.

Indicator.—A mechanical device that indicates the movements of water or water controlling devices.
See, also, *Register; Recorder.*

Infiltration.—The percolating flow of ground-water into a drain, or other covered conduit.

Inlet.—(1) A surface connection to a closed drain; (2) a structure at the diversion end of a conduit; (3) the up-stream end of any structure through which water may flow.

Intake.—The head-works of a conduit; the point of diversion.

Intensity of Pressure.—The pressure per unit area.

Intercepting Channel.—A channel excavated at the top of earth cuts to intercept surface flow and protect the cut slopes from erosion.

Invert.—The floor, bottom, or lowest part of the internal cross-section of a conduit.

Inverted Siphon.—A pipe line crossing a depression. The term is common but inappropriate as no siphonic action is involved.

Irregular Weir.—A weir whose crest is not of standard or regular shape.

Irrigable Area.—The area under an irrigation system capable of being irrigated principally as regards quality and elevation of land. It generally includes roads, farm lots, building sites, and miscellaneous areas not actually irrigated.

Irrigating Head.—(1) The flow used for irrigation of a particular tract of land; (2) the quantity of water handled by a single irrigator, or that in a single farm lateral.

Irrigation.—The artificial application of water to lands for agricultural purposes.

Irrigation District.—An organization of land owners operating under legal regulations for financing, constructing, and operating an irrigation system.

Irrigation Water.—The quantity of water artificially applied in the processes of irrigation. It does not include precipitation.

Irrigation Requirement.—The quantity of water, exclusive of precipitation, that is required for crop production. It includes economically unavoidable wastes.

Irrigator.—One who applies water to land for growing crops.

Jetty.—A stone dike or other structure built in a stream to induce scouring of one portion of the channel and building up of another by silting.

Jump.
See *Hydraulic Jump.*

Kinetic Energy.—The energy of flowing water due solely to its motion. It is proportional to the product of flow and velocity head.

Kutter's Formula.—A formula for the value of the coefficient, C, in the Chezy formula, the factors of which are the friction slope, the hydraulic radius, and a coefficient of roughness.

Laminar Flow.—Flow in layers; that is, stream-line flow in contradistinction to turbulent flow.

Lateral.—(1) A conduit diverting water from a main conduit, for delivery to distributaries; (2) a secondary ditch.

Lateral-Flow Spillway.—A spillway in which the initial and final flow are approximately at right angles to each other; a side-chanel spillway.

Leach.—To remove alkali from soils by abundant irrigation coupled with drainage.

Left Shore or Bank of a Stream.—The left-hand shore when one is looking down stream.

Length-of-Run.—The distance water must run in furrows or over the surface of a field from one head ditch to another, or to the end of a field.

Levee.—A dike or embankment for the protection of lands from inundation, or for the purpose of confining stream flow.

Leveler.—A buck scraper, drag, or any other form of device for smoothing land for irrigation.

Lining.—A protective covering over all, or over a portion, of the perimeter of a canal, tunnel, or reservoir, to prevent seepage losses, to withstand pressure, or resist erosion.

Log Chute, Log-Way.—A by-pass around or through a dam for logs and drift.

Lost Head.—The energy converted into heat as a result of friction, eddies, and impact, expressed as the height of a column of water whose potential energy is equivalent to that loss.

Machine-Banded Pipe.—Wooden pipe made in certain lengths and joined in the field by couplings. The staves are wrapped by wire, in a machine.

Main Canal.—The main conduit beginning at the source of water supply, from which the lateral system receives its supply.

Manning's Formula.—A formula for the value of the coefficient, C, in the Chezy formula, the factors of which are the hydraulic radius and a coefficient of roughness.

Manometer.—A tube containing a liquid, the surface of which moves proportionally to changes of pressures; a **U**-tube; a tube type of differential pressure indicator.

Mass Diagram.—A graphical representation of accrued quantities, as the integral of a time-flow curve. In a mass curve each point is the sum of all preceding quantities considered. It is used extensively in storage problems.

Mean Depth.—Cross-sectional area of water prism divided by surface width.

Mean Velocity.—The velocity obtained by dividing flow by cross-sectional area of water prism.

Measuring Weir.—A weir constructed for the purpose of measuring flow. It consists of a rectangular, trapezoidal, triangular, or other shaped notch in a comparatively thin plate whose plane is usually vertical and through which the water flows.
See, also, *Cipolletti Weir; Triangular Weir.*

Miners Inch.—A stream of water from an orifice 1 in. square under a definite head, used as a unit of measurement and independent of time. The value of a miners inch has been fixed by statute in various States as

follows: In Arizona, California, Montana, and Oregon, 40 miners in. are the equivalent of 1 cu. ft. per sec.; in Idaho, Nebraska, Nevada, New Mexico, North Dakota, South Dakota, and Utah, 50 miners in. are the equivalent of 1 cu. ft. per sec.; in Colorado, the accepted equivalent is 38.4 and, in British Columbia, 35.7. In some parts of California engineers use 40 miners in. to 1 cu. ft. per sec., while in the southern part quite generally they use 50 miners in. to 1 cu. per sec., regardless of the statute definition.

Module.—A device for delivering a definite quantity of water, or for measuring the flow of water.

Movable Dam.—A dam that may be opened in whole or in part. The movable portion may consist of gates, stop-logs, needles, wickets, or any other device whereby the area for flow through or over the dam may be controlled.

Mulch.—A loose covering of straw, manure, sand, dust, etc., to reduce evaporation.

Multiple-Arch Dam.—A dam consisting of a series of arches supported by transverse buttresses. The load is transferred by the several arches to the foundation through the buttresses.

Nappe.—A sheet or curtain of water overflowing a weir, dam, etc. The nappe has an upper and a lower surface.

Needle.—A timber set on end to close an opening for the control of water; it may be either vertical or inclined; a form of stop-plank.

Net Duty of Water.—The irrigation water applied to a farm unit.
See, also, *Duty of Water.*

Neutral Depth.—The depth of water in an open conduit that corresponds to uniform velocity for the given flow. It is a hypothetical depth under conditions of steady, non-uniform flow; the depth for which the surface and bed are parallel.

Non-Uniform Flow.—A flow whose quantity or velocity is undergoing a positive or negative acceleration. If the flow is constant it is referred to as "steady non-uniform flow."

Normal Depth.—(1) The distance between the water surface and the bed of a stream measured perpendicular to the stream bed; (2) the average depth; and (3) the neutral depth.

Normal Year.—A year of normal or average water supply.

Notched Weirs.
See *Measuring Weir.*

Off-Take.—A diversion.

Ogee.—The reversed curve of the face of an overflow dam, named after the ancient letter, S.

Optimum.—The most favorable for the results desired; as, for instance, the optimum quantity of irrigation water, the optimum soil moisture, etc.

Orifice.—An opening through which water may flow.

Outfall.—The point where water flows from a conduit; generally restricted to drains and sewers.

Over-Fall.—(1) The portion of a dam or weir over which the water pours; (2) the over-pouring water.

Overflow Stand.—A stand-pipe in which water rises and overflows at the hydraulic grade line.

Overhaul.—The transportation of excavated material beyond certain specified limits.

Parabolic Weir.—A measuring weir whose notch is bounded on the sides by parabolas such that the flow is proportional to the head.

Parshall Measuring Flume.—A device developed by the U. S. Department of Agriculture and the Colorado Experiment Station, at Fort Collins, Colo., under the direction of Ralph L. Parshall Assoc. M. Am. Soc. C. E., Senior Irrigation Engineer, to measure the flow of water; formerly called the "Improved Venturi Flume." [3]

Penstock.—A pressure conduit for supplying water to a water-wheel or turbine.

Percolation.—Movement of water through the interstices of a substance, as through soils.

Permissible Velocity.—The highest velocity at which water may be carried in a canal or other conduit without injury thereto. The highest velocity throughout a substantial length of a conduit that will not scour.

Piezometer.—An instrument for measuring pressure head, usually consisting of a small pipe tapped into the side of a closed or open conduit and flush with the inside, connected with a pressure gage, mercury, or water column, or other device for indicating pressure head.

Pitot Tube.—A device for observing the velocity head of flowing water, consisting essentially of an orifice held up stream in a prism of flowing water and connected with a tube by which the rise of water above the water surface may be observed. It may be constructed with an up-stream and a down-stream orifice and two water columns, the difference of water levels being an index of the velocity head.

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[3] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 841; also, Colorado Experiment Station *Bulletin 336*.

Plant Consumption.—The water used by plants in the processes of growth. It includes that stored in the body of the plant and that dissipated from its leaf and body surfaces by transpiration.

Porosity.—A measure of the "openness" of soils permitting percolation.

Potential Energy.—Energy due to position. The potential energy of a given flow with reference to any datum is represented by the product of the elevation of its water surface above that datum, and its weight.

Precipitation.—The total measurable supply of water received directly from clouds, whether in the form of rain or snow; usually expressed as depth in a day, month, year, and designated as daily, monthly, annual precipitation.

Pressure.—Total load or force acting upon a surface; frequently used to indicate intensity of pressure or pressure per unit area.

Pressure Head.—The head on any point in a conduit represented by the height of the hydraulic grade line above that point.

Priming.—(1) (Noun) The first filling of a canal or other structure; that is, either the absolutely first, or the seasonally first; (2) (Verb) starting flow as in a pump or siphon.

Puddle.—(1) (Noun) Material placed with water to form a compact mass to reduce percolation; (2) (Verb) to place such material.

Race.—The channel which leads water to or from a water-wheel; the former is "head-race," the latter, "tail-race."

Rack.—A screen composed of parallel bars to arrest floating débris.

Radial Gate.—A pivoted gate whose face is a circular arc with center of curvature usually at the pivot.

Rainfall.—Precipitation in the form of water. Usage has countenanced the inclusion of snowfall in the term.

Ram.
See ***Hydraulic Ram.***

Rapids.—A term used by some writers for "chute".

Rating.—The relation empirically determined between stage and flow of water. The taking of measurements at a given station by current meter or otherwise to determine this relationship is termed "rating the station".

Rating Curve.—A graphic representation of the relation between stage and flow.

Rating Flume.—A flume built in a channel to maintain a constant regimen for the purpose of measuring the flow and establishing a rating.

Reconnaissance.—A preliminary examination of a proposed project.

Recorder.—A mechanical device that makes a record of the movements of water or water-controlling devices.
See, also, *Indicator: Register.*

Recording Gage.—A recorder that makes a hydrograph.

Rectangular Weir.—A contracted measuring weir whose notch is rectangular in shape. A suppressed weir is generally rectangular in shape, but is not included in the term "rectangular weir."

Register.—A mechanical device that either indicates or records, or both.
See, also, *Recorder: Indicator.*

Reservoir.—A pond, lake, or basin, either natural or artificial, for the storage, regulation, and control of water.

Ridging.—Making small embankments or borders in fields to control irrigation water.

Right Shore or Bank of a Stream.—The right-hand shore when one is looking down stream.

Riparian.—Pertaining to the banks of a body of water; riparian owner is one who owns the banks; riparian right, the right to control and use water by virtue of ownership of the bank or banks.

Riprap.—Broken stone placed on earth surfaces for their protection against the action of water; also, applied to brush or pole mattreses, or brush and stone or other similar materials used for protecton.

Rock-Fill Dam.—A dam composed of loose rock usually dumped in place often with the up-stream face of hand-placed rock and faced with rolled earth; or an impervious diaphragm of concrete, timber, or steel.

Rod-Float.—A rod or staff designed to float in a practically vertical position for the purpose of observing velocities.

Roller-Gate.—A hollow cylindrical gate with spur gears at each end meshing with an inclined rack anchored to a recess in the end pier or wall. It is raised or lowered by being rolled on the rack.
See, also, *Sector Gate.*

Rollway.—The overflow portion of a dam; an overflow spillway.

Root Zone.—The stratum of soil invaded by the roots of plants.

Rotary Pump.—A displacement pump for raising a liquid by the use of rotating elements instead of by pistons.

Rotation.—A system of irrigation through which the irrigator receives his allotted quantity of water, not at a continuous rate, but as a large flow at stated intervals; for example, a number of irrigators receiving water from the same lateral may agree among themselves to rotate the water, each taking the entire flow in turn for a limited period.

Run-Off.—The portion of precipitation that appears as flow in streams.

Run-Off Coefficient.—The ratio of run-off to precipitation.

Sand-Trap.—A device in a conduit for arresting the sand and silt carried by the water, including a means of discharging the sand from the conduit.

Scouring Sluice.—An opening in a dam controlled by a gate through which the silt, sand, and gravel accumulated may be periodically discharged.

Screen.—A curtain with openings to catch floating débris, control movements of fish, etc.

Second-Foot.—A cubic foot per second; a cusec.

Second-Foot-Day.—The volume of water represented by a flow of 1 cu. ft. per sec. for 24 hours. It is 86 400 cu. ft., or practically 2 acre-ft., a convenient unit in storage computations.

Sector Gate.—A roller type of gate in which the roller is a sector of a circle instead of a complete cylinder.
See, also, *Roller Gate.*

Seepage.—The flow of water from a canal or reservoir by percolation, expressed as depth over the surface or wetted perimeter in a given time.

Self-Mulching Soil.—A soil that breaks up into fine, dry dust by cultivation, forming a mulch.

Semi-Arid.—A term applied to a country not entirely arid nor strictly humid, but intermediate. A dry-farming country in which many crops grow without irrigation, but in which far better yields result from irrigation.

Settling Basin.—An enlargement in a conduit to permit the settlement of débris carried in suspension, usually provided with means of ejecting the material so collected.

Sheet-Piling.—A diaphragm of wood, steel, or concrete, driven to form an obstruction to percolation or to prevent material from caving into an excavation.

Shingle.—Gravel and cobblestones deposited by water to resemble lapped roofing pieces. The origin is "shingl"—a Norwegian term for a small round stone.

Side-Channel Spillway.
See ***Lateral-Flow Spillway.***

Side Slopes.—The slope of the sides of a canal, dam, or embankment; custom has sanctioned the naming of the horizontal distance first as 1.5 to 1, meaning a horizontal distance of 1.5 ft. to 1 ft. vertical.

Silt.—Fine material carried by flowing water; sediment. In India the term includes all detritus transported, even coarse gravel.

Siphon.—A closed conduit curved in profile, which utilizes atmospheric pressure to effect or increase the flow of water through it. (An inverted siphon has none of the properties of a siphon—the term is a misnomer.)

Skew-Frequency Curves.—A frequency or probability curve that is unsymmetrical about the ordinate of maximum frequency.

Skimming.—Diverting surface water by shallow overflow to avoid diverting sand, silt, or other débris carried as bottom load.

Sluice.—(1) (Noun) conduit for carrying water at high velocity; (2) (Noun) an opening in a structure for passing silt; (3) (Verb) to cause water to flow at high velocities for wastage, for purposes of excavation, etc.

Slush Ice.
See ***Frazil Ice.***

Snow Course.—A course laid out and permanently marked on the headwaters of a stream, along which the snow is regularly sampled to determine its depth and density, the object being to forecast subsequent run-off.

Snow Density.—The water content of a given snow sample, usually expressed as water depth in percentage of snow depth; that is, the depth of water the snow would make if melted.

Snow Sample.—A core cut from a snowbank on a snow course, from which the depth and density may be determined.

Snow Sampler.—A set of light jointed tubes for taking snow samples and a spring scale graduated to read directly the corresponding depth of water contained in a snow sample.

Snow Surveys.—Surveys usually made in the spring of the year on the headwaters of streams to determine the water stored thereon in the form of snow, as a means of forecasting the subsequent spring run-off.

Soil.—Finely divided material composed of broken down rock mixed with decaying plant and animal remains.

Soil Evaporation.—Evaporation of water from moist soils.

Soil Moisture.—The quantity of water held within soil available for crop production.

Spillway.—A passage for spilling surplus water.

Spur-Dike.—A dike of rock or other structure built from the bank into the channel for bank protection.

Staff Gage.—Graduations on a staff, plank, pier, wall, etc., by which the elevation of a water surface may be read.

Stage.—The elevation of the water surface of a stream above its minimum.

Standing Wave.—A sudden rise in the water surface; a limiting case of a surface curve where it becomes vertical; a hydraulic jump. A standing wave may exist, however, where the principles of the hydraulic jump are not involved.

Stand-Pipe.—A pipe or tank connected to a closed conduit and extending to or above the hydraulic grade line.

Steady Flow.—A constant flow, that is, the same volume in equal units of time.

Stilling Well.—A chamber closed at the bottom and connected to an adjoining body of water by a small inlet, with the object of stilling or damping wave action therein.

Stop-Log.—A heavy log, plank, cut timber, steel or concrete beam fitting into end guides between walls or piers to close an opening to the passage of water; usually handled one at a time.

Storage Dam.—A dam designed to store water during periods of surplus stream flow for use during periods of lower flow.

Stream.—A prism of flowing water. This term is sometimes, but incorrectly, used to indicate the channel in which the prism flows.

Stripping.—Removing the top layer containing vegetable matter or other stratum containing undesirable material.

Subbing.—Sub-irrigating.

Sub-Drain.—A drain built below the grade of a pipe, culvert, or the lining of a canal, to carry off the seepage.

Sub-Irrigation.—(1) Watering plants below the ground surface; (2) irrigation by a rising ground-water plane, or by control of the water-table.

Submerged Orifice.—An orifice which in use has the tail-water above the crown of the orifice.

Submerged Weir.—A weir which in use has the tail-water level equal to or higher than the weir crest.

Submergence.—The ratio of the tail-water elevation to the head-water elevation, using the crest of the overflow structure as datum, when both head-water and tail-water are at higher elevations than the crest. The distances up or down stream from the crest at which head-water and tail-water elevations are measured are important, but cannot be prescribed.

Subsoil.—That soil lying below the surface soil.

Sub-Surface Float.—A submerged body which is attached by a line to, and whose movement is indicated by, a surface float; used for the purpose of observing velocities.

Suppressed Weir.—A measuring weir the sides of whose notch are flush with the channel, thus eliminating (suppressing) end contractions of the overflowing water. A weir may be suppressed on one end or on two ends. If bottom contractions are suppressed, it ceases to be a measuring weir and becomes more in the nature of a control.

Surface Curve.—(1) The longitudinal curve assumed by the surface of water flowing in an open conduit; the surface curve is the curve of equilibrium of all forces acting on the flowing water; (2) the hydraulic grade line.

Surface Evaporation.—Evaporation from the surface of a body of water, snow, or ice.

Surface Float.—A float on a water surface used to indicate velocity or direction of flow.

Surface Slope.—The inclination of the water surface expressed as change of elevation per unit length; the sine of the angle which the water surface makes with the horizontal. The tangent of that angle is ordinarily used, no error being involved, except for the steeper slopes.

Tail-Race.—A channel conducting water away from a water-wheel.

Tail-Water.—The water just down stream of a structure.

Taintor Gate.
See *Radial Gate.*

Test Pit.—A hole excavated to determine the nature of the material encountered, or to disclose sub-surface conditions.

Tilting Gate.—A gate hinged at top or bottom and counterbalanced by weights, automatically opening or closing with the change in head.

Transition.—A short conduit uniting two others having different hydraulic elements; a conversion.

Transpiration.—The process by which plants dissipate water from their leaf and body surfaces.

Transpiration Ratio.—The ratio of the weight of water passing through a plant, to the weight of dry plant substance produced.

Trapezoidal Weir.—A contracted measuring weir whose notch is trapezoidal in shape.
See, also, ***Cipolletti Weir.***

Traveling Screen.—A diaphragm, usually of canvas in a frame moved by water in the direction of flow, for purposes of measuring directly the mean velocity; only useful in regular channels where the frame is shaped to the channel cross-section and nearly fills it.

Triangular Weir.—A measuring contracted weir the sides of whose notch are the sides of an angle with apex downward, the crest being the apex of the angle; a V-notch weir.

Underflow.—Movement of water below the surface of the earth or beneath a structure; the flow of percolating water, of water under ice, or under a dam.

Uniform Flow.—A flow whose velocity is constant. If the flow is also constant it is referred to as "steady uniform flow."

Uplift.—The aggregate pressure on the base of a structure due to the head of water on the foundations.

V-Notch Weir.
See ***Triangular Weir.***

Velocity.—Rate of movement in relation to time.
See, also, ***Mean Velocity: Average Velocity.***

Velocity Head.—For a given velocity, the distance a body must fall under the force of gravity to acquire that velocity. The kinetic energy of flowing water is proportional to the velocity head.

Velocity of Approach.—The mean velocity of water approaching a structure. It has a particular significance for weirs, orifices, and other measuring devices, wherein the velocity of approach must be taken into account.

Velocity of Recession (Retreat).—The mean velocity of the water as it leaves a structure.

Vena Contracta.—The contracted vein.
See, also, ***Contraction.***

Venturi Flume.—A type of open flume with a contracted throat that causes a drop in the hydraulic grade line; used for measuring flow. One type has been developed by the U. S. Department of Agriculture and the Colorado Agricultural Experiment Station at Fort Collins, Colo.
See, also, ***Parshall Measuring Flume.***

Venturi Meter.—A closed conduit in which a gradual contraction is placed, which causes a reduction of pressure head by which the velocity may be determined. The contraction is generally followed, but not necessarily so, by an enlargement to original size. The section of smallest area is called the throat; the approach to the throat, the contracting section; and the recess from the throat, the expanding section if it exists. Named after G. B. Venturi, an Italian physicist, and developed and patented by the late Clemens Herschel, Past-President and Hon. M., Am. Soc. C. E.

Vertical-Velocity Curve.—The relation between depth and velocity expressed graphically.

Wasteway.—The channel required to convey water discharged into it from a spillway, escape, or sluice.

Waste Weir.—A spillway.

Water Cost.—The quantity of water used for any particular purpose; for example, the water cost of plant growth is expressed by the transpiration ratio. Water demand is a better term.

Water Cushion.—A pool of water maintained to take the impact of water overflowing a dam, chute, drop, or other spillway structure.

Water Film.—(1) The film of water surrounding and held by a soil particle; (2) any thin layer of water.

Water-Hammer.—Pressure in excess of static pressure in a pipe line due to change of momentum of the flowing water. The pressure resulting from water-hammer is oscillating.

Water-Level Recorder.—A device for producing a graphic record of the rise and fall of water with respect to time.

Water-Logging.—Over-irrigation of lands until the ground-water rises to a level detrimental to plant growth.

Water Requirement.—The total quantity of water, regardless of its source, required by crops for their normal growth under field conditions. See, also, *Irrigation Requirement.*

Water Right.—A legal right to the use of water.

Water-Shed.—The area drained by a stream or stream system.

Water-Table.—The plane of free water in saturated soil below the ground surface.

Water-Year.—A special grouping of the periods of a year to facilitate water supply studies. The U. S. Geological Survey uses October 1 to September 30.

Weep-Holes.—Openings left in masonry to permit drainage.

Weir.—A dam across a stream for diverting or for measuring the flow. As applied to a diversion dam this term is gradually becoming obsolete in the United States.
See, also, ***Measuring Weir.***

Wetted Perimeter.—The length of the wetted contact between the water prism and the containing conduit, measured along a plane at right angles to the conduit.

Wicket-Dam.—A movable dam made up of wickets, or shutters, revolving about a central axis.

Wilting Coefficient.—The soil moisture in percentage of dry weight remaining in the soil within the root zone as plants reach a condition of permanent wilting.

Wing-Dam.—A dam projecting from the shore of a stream; a spur-dike.

Winter Irrigation.—The irrigation of lands during the non-growing season in order to store water in the soil for subsequent use by plants.

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DISCUSSIONS

MANUFACTURING CONCRETE OF UNIFORM QUALITY

Discussion

BY MESSRS. W. B. WENDT, AND WILLIAM M. HALL

W. B. WENDT,[23] M. AM. SOC. C. E. (by letter)[23a]—This discussion is confined to tests made at the laboratory of the University of Louisville, begun in May, 1928, as presented in Table 1 of the paper. The compressive strength of the 1929 concrete averaged the highest for the five years and the variations of Item 3 from Item 2 and of Item 4 from Item 2 (Table 1) are considerably less for 1929 than for any other year, as shown in Table 6.

TABLE 6.—PERCENTAGE VARIATION IN STRENGTHS

Items corresponding to Table 1	1925	1926	1927	1928	1929
TWENTY-EIGHT DAY TESTS (VARIATION ABOVE THE AVERAGE)					
Item 2	2 157	2 719	2 685	2 691	4 779
Item 3	1 590	2 186	2 128	2 215	4 308
Difference	567	533	557	476	471
Percentage difference	26	20	21	18	10
TWENTY-EIGHT DAY TESTS (VARIATION BELOW THE AVERAGE)					
Item 4	2 933	3 217	3 302	3 326	5 186
Item 2	2 157	2 719	2 685	2 691	4 779
Difference	776	498	617	635	407
Percentage difference	36	18	23	24	9

The percentage differences in Table 6 show conclusively that the quality of the 1929 concrete was much more uniform than that of other years, and that the continued insistence on more uniform and better concrete on the part of inspectors on the job was bringing results. As the writer

NOTE.—The paper by William M. Hall, M. Am. Soc. C. E., was published in May, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. Henry B. Seaman, Theodore Belzner, John Sanford Peck, and J. W. Kelly; November, 1931, by Messrs. Stanton Walker and Edward E. Bauer; December, 1931, by Messrs. O. Bonney and D. T. Mitchell, C. E. Arnold, and Morris Mason; and March, 1932, by John H. Griffith, M. Am. Soc. C. E.

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[23a] Received by the Secretary March 28, 1932.

understands it, one of the main objects of Table 1 was to point out that good concrete could be obtained and that it would be fairly uniform.

The nine jobs were spread out for nearly 400 miles, and concrete was made by different organizations using different types of equipment and a variety of cements, admixtures, and river-bar aggregates. It is because of these conditions that the small variation in test results for the year 1929 is remakable.

Capping and Storing the Cylinders.—The method of rocking a cylinder by hand on a plane surface (or by the "feel") as reported in Appendix I, was not the only method used to determine whether the ends of the test cylinders were truly plane surfaces. At the laboratory, the rocking of the cylinder was only one of several tests to which the cylinder was subjected to determine whether the ends were satisfactory for testing. Each cylinder was given a thorough visual inspection and each was also subjected to a steel straight-edge, when considered necessary. The cylinders were tested with a spherical bearing-block in the testing machine, and, therefore, no error was caused by eccentric pressure due to any ends not being parallel.

Contrary to current belief it is quite possible to apply the plaster-of-Paris cap without rocking the plate or the cylinder in such a way as to produce a curved surface. The method of procedure is as follows: The plaster of Paris is mixed with water to produce a paste of very plastic consistency. The correct quantity (determined by experience) of this paste is applied to one end of the test cylinder. This end is immediately placed on the bottom machined plate, and the cylinder is rotated and pressed down on the plate by hand until the thickness of the plaster of Paris is about $\frac{1}{8}$ in., or less. The paste is then applied to the upper end of the cylinder. A machined plate is immediately placed on this upper end of the test specimen and the cylinder pushed into position in the testing machine with the upper and lower machined plates still in place. The moving head of the testing machine is brought into contact with the upper plate, a load of several hundred pounds is applied to the test specimen, and this load is kept on until the plaster of Paris becomes hard, requiring at least 15 min. The cylinder is then tested to failure. The writer has examined several hundred of these plaster-of-Paris caps after removing the test specimen from the machine, and he has never found one with a crack that indicated any signs of failure.

Cylinders should not be discarded because the ends of the test cylinder are not parallel. They are made in the field in an endeavor to ascertain the strength of the concrete. When these cylinders reach a testing laboratory, the laboratory technician must test them, using every precaution to insure that the test results will have value. The writer used the plaster-of-Paris cap in an effort to secure pressure distribution over the entire surface of the test specimen. All the testing machines used are equipped with spherical bearing-blocks, because it is hopeless otherwise to expect results to have any value. The test results on pairs of cylinders made of the same concrete check remarkably well, and it might be well actually to compute the variation from the mean of each pair of cylinders in order to show just how close the check was.

In many pairs the breaks of the two cylinders were remarkably close, and, in fact, two pairs under loads of nearly 200 000 lb. were recorded as nearly equal as could be read on the pulling arm, 5 ft. long, the sensitivity of which was 10 lb.

All cylinders received from May, 1928, were tested either by W. R. McIntosh, Assoc. M. Am. Soc. C. E., or by the writer.

William M. Hall,[24] M. Am. Soc. C. E. (by letter).[24a]—It is gratifying to note that there is much unanimity of opinion in reference to the advisability of teaching and training inspectors (even after graduation from the best engineering schools) as to "job" methods, system, and technique, and the treatment and training of the men who do the real work of delivering, measuring, and mixing materials, and transporting, placing, and curing the concrete.

The writer agrees with Mr. Seaman in his statement as to quality requiring "more than uniformity," and that it should be of uniform good quality. It is desirable to secure such uniformity for every batch of concrete, and the record thereof, as indicated by every cylinder, should be made. All the details he names—thorough mixing, sufficient caution in "moulding" or placing to overcome the segregation resulting from transportation, thorough "spading" next to forms to give a mortar face on every stone or gravel, and thorough spading in the body to release the bubbles of air entrapped when dumping—are desirable or requisite. Such uniform care day after day, month after month, and year after year, can scarcely be attained except by selecting those inspectors and operatives in whom some pride and enthusiasm can be inspired for accomplishing good work, or better work than that which is just passable. When forms are removed, each man trained for his particular work, or part of the work of each monolith, should make an inspection of the finished work with a diagram in hand, showing the engineer in charge the part done by each. If systematically done, this tends to perfect such good accomplishment and interest in the finished work.

Possibly, Mr. Peck's exception to the statement that 50% of the concrete made to-day has a short life, is well taken. It is to be hoped that such is the case, but that 50% is below the average quality is believed to be unquestionable, and much below the quality of most of the concrete tested and reported in Table 1, for the four years, 1926 to 1929, inclusive. A careful reading of Mr. Peck's comments following that statement tends to confirm the accuracy of the writer's estimate of poor concrete. Possibly 50% is a little high and possibly Mr. Peck has not seen so much concrete, not only poor, but failing, as it has been the writer's misfortune to see.

As Mr. Kelly, who had some years of association with Professor Abrams, was the principal and invaluable aid in conducting the training school for inspectors, he fully understands its importance, and, on large structures, the importance of perfecting and systematizing methods of directing the workmen and conduct of the work by any new assembly of inspectors, such service as he rendered is almost invaluable.

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[24] U. S. Civ. Engr., Parkersburg, W. Va.

[24a] Received by the Secretary March 28, 1932.

Mr. Walker's exception to the specification of "15% or more of mortar than was needed to fill the voids of the gravel," is well taken. The actual practice throughout all the work was (as stated in Appendix II under "Batch Mixing,") "the specified measure of sand (equal to voids in gravel plus 15% of voids)" and, in addition, due allowance was made for bulking where an inundator was not used in measuring the sand. In fact, emphasis was always placed on the importance of preventing any misunderstanding and of always having uniformly workable concrete. It is known that with the mortar specification in force, unless the sand carries an unusual quantity of fines, or unless an admixture is used, a harsh product may result. Therefore, the "plus 15%" of sand was the adopted practice, emphasis being placed on "workability."

Professor Wendt submits his comments on laboratory practice, in breaking test cylinders made on this work. The answer to the questions raised by Messrs. Kelly, Walker, and Bauer and by Professor Griffith will be found in this discussion. It is observed that the impression was given that the method

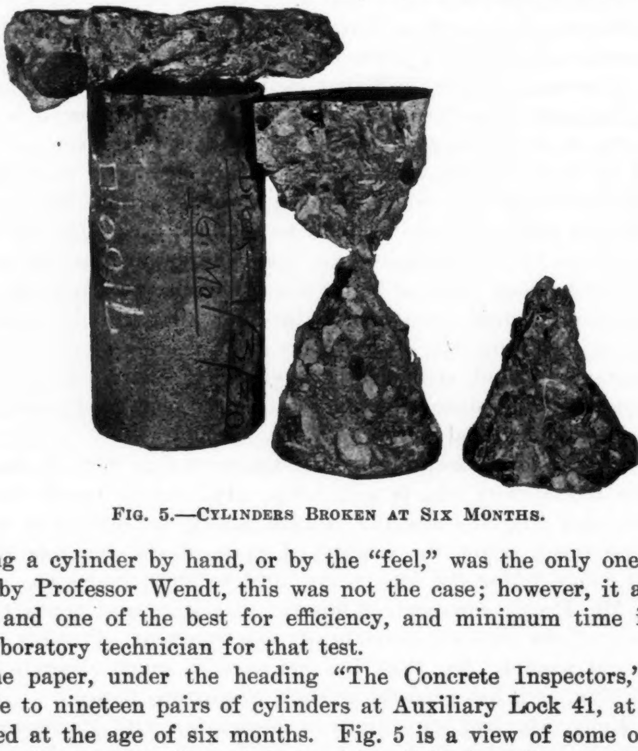

FIG. 5.—CYLINDERS BROKEN AT SIX MONTHS.

of rocking a cylinder by hand, or by the "feel," was the only one used. As affirmed by Professor Wendt, this was not the case; however, it appears the simplest and one of the best for efficiency, and minimum time is required by the laboratory technician for that test.

In the paper, under the heading "The Concrete Inspectors," reference was made to nineteen pairs of cylinders at Auxiliary Lock 41, at Louisville, and tested at the age of six months. Fig. 5 is a view of some of these job cylinders after testing (from left to right, Cylinders Nos. 54, 53, and 71). Cylinders Nos. 53 and 54, tested at the age of six months, were subjected first to 200 000-lb. pressure without breaking. A month later Cylinder No. 53 broke at a pressure of 194 000 lb. Cylinder No. 71 broke, at six months, under a pressure of 182 820 lb., as reported in the paper. Note the numerous sheared pieces of gravel.

It is gratifying to have such an extensive exchange of experience on the important details of laboratory technique; however, the writer will add a few explanatory words. As an example of the practice, Appendix I describes only the making and finishing of cylinders in laboratories "on the job." Professor Wendt's remarks are all in reference to the treatment after the cylinders are transferred from the field to the University Laboratory—two entirely different parts of the process by different personnel. The writer has emphasized the importance of completing the cylinders entirely and curing them "on the job" to within about five days of breaking, thereby reducing the chance of fracture in transfer to a minimum. After arrival at the Laboratory, it was the exception for a cylinder to require more than an examination of the ends and its general condition. It was then immediately placed in storage for curing until the hour of breaking. Each cylinder requiring new plaster-of-Paris ends consumed from 15 to 30 min. additional. Hence, the detailed description of the two processes, and the effort to prevent re-capping with plaster of Paris.

Preceding the engagement with Professor Wendt for breaking the cylinders, all the other cylinders included in Table 1, were broken by or under the personal direction of Herman H. Heck, Chemical Engineer, in a laboratory equipped and operated by the Byllesby Engineering and Management Corporation. Mr. Heck states that the methods of treatment, and the use of spherical head for applying and equalizing pressure conformed throughout with Professor Wendt's procedure. As regards finishing the ends of cylinders in field laboratories, it may be said in full justice to Mr. Little, that he has tried numerous methods of perfecting cylinders and, compared with the others, the "feel" appears to be the final test in his field laboratory for giving the best results, and the writer has observed no better method.

Messrs. Bonney and Mitchell who give such a good record of work accomplished for the City of Columbus, Ohio, are fortunate in the accomplishment recorded in Table 4. Mr. Arnold and the Los Angeles land owners are nearly as fortunate in obtaining concrete of about the same uniformity as that in Table 4, although with not quite so high an average strength. As the drains described are not subjected to freezing (which makes the question of waterproofness a minor factor of design), it appears that, should the uniformity in strength of the concrete be maintained it would be good practice to reduce the measure of cement to produce concrete slightly exceeding the designed strength. In fact, that is one of the most important reasons for such tests. It is regrettable that many other records of such uniform concrete were not given by other engineers or contractor managers. It adds to the evidence that such uniformity is only a question of training engineers, inspectors, and workmen, and that when owners become informed as to the increased durability of their structures, built of concrete of such uniform quality, it is probable that such methods and results will be demanded by them.

The writer wishes to express sincere appreciation and thanks to all who have contributed to this paper, including all those who took part in the construction described, and especially to all those with whom he was intimately associated in any part of the construction or testing work.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FINANCING STREET AND HIGHWAY IMPROVEMENTS

Discussion

By Horace H. Sears, M. Am. Soc. C. E.

Horace H. Sears,[10] M. Am. Soc. C. E. (by letter).[10a]—A serious phase of engineering design and construction is discussed in this paper. Future financing of these improvements is dependent upon the sale to private investors of millions of dollars in new and refunding securities. These securities are issued by highway districts and special assessment districts in the form of bonds, notes, and warrants. Each of the securities is a promise to pay by the district, as a corporation, for these improvements. The improvements become the property of the district when installed; yet they cannot be attached or sold to satisfy the costs for labor and materials as is the case in constructing improvements for private corporations.

The intent of this discussion is to call the attention of engineers to the need of recognizing the financial limitations of these highway and special assessment districts under the limited powers which the Legislatures of various States have conferred upon them. The design of roads, the appropriation budget, and the prospect of securing funds are radically different for the city special assessment district as compared with the county road district or the State highway district. It is of no avail to design highways unless funds are permissible under highway district powers and unless authorized securities can be sold to investors. The payment for construction and for engineering services comes from the sale of corporate securities. Repayment of these loans must be assured or no funds will be forthcoming.

The sale of securities for city street improvements is best assured when bonds are voted and payable from *ad valorem* taxes. Preliminary engineering designs and costs must be determined in advance of the actual authorization of these voted bonds. To-day, more than previously, the city special assessment district is a corporate form of State agency which must give the

Note.—The paper by R. W. Crum, M. Am. Soc. C. E., was published in August 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1931, by Messrs. A. P. Greensfelder and C. R. Thomas; February, 1932, by J. C. Carpenter, M. Am. Soc. C. E.; and April, 1932, by Samuel Eckels, M. Am. Soc. C. E.

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[10a] Received by the Secretary March 7, 1932

engineer cause for thought. The sale of such securities for many cities meets with a great deal of sales resistance from the investing public. The city credit may be stated on the bonds, but experience shows that the repayment to the investor is dependent upon collection of assessments against the property in the district benefited.

The sale of securities for county road districts and for State highway district improvements is direct to the purchaser, without the intermediary of a city to supervise collections. This form of security is dependent for repayment, in the event of delinquent tax collections, upon foreclosure proceedings against delinquent property in the district. These district corporations are agencies of the State, created for a special purpose by the Legislature. They have limited powers in the matter of issuing securities, which are, therefore, not as readily sold to investors as the *ad valorem* tax bond voted by cities. In view of the present financial stringency, the design of renewals, repairs, and new road construction requires a knowledge of the probable financing of construction in the present securities market, as well as a knowledge of the corporate powers of the district. The legislative powers may be examined by attorneys for the engineer, but if funds are not forthcoming in the desired amount, the design and budget should be shaped accordingly. The resourceful engineer will prove his value to the community in maintaining traffic under these limited sources of funds. Highway districts must economize as many such corporations have reached the statutory debt limit.

District financing and special assessment financing for cities are very different so far as securing funds from the investor at the present time (1932). The engineer should investigate the terms, "highway district bond," and "special assessment bonds," and note the exact promise of the debtor corporation. Funds are secured at less cost when the credit of a city is back of the special assessment bond, than in the case of the district bond. The district road warrant, note, or tax anticipation certificate, should be studied with care. These securities are restricted, as a general rule, to payment for an existing indebtedness. When issued for loans and based on the future credit of the district, the enabling act of the Legislature should receive special attention from attorneys who specialize in municipal securities.

The investor has learned, and the engineer should know, that the district corporation is termed in law a quasi-municipal corporation, which is of limited character when compared with the municipal corporation, which is usually designated as the city. An approving opinion by a bond attorney, on procedure for a district security proposed to be issued, is in the nature of a preliminary statement. The final approving opinion of the attorney may not be given. Engineering designs, both preliminary and final, may be tied up with insufficient funds because of the failure to secure a final approving opinion, or because litigation is pending over the proposed issue of bonds. Highway construction may be imperatively needed, but if funds are not forthcoming the traveling public may detour for months because of incomplete paving. In such cases alternate highway plans must be provided until securities are sold.

The need for further assurance in this matter of financing by sales to the investor of special assessment securities and of district bonds has resulted in the recent publication by The Investment Bankers Association of America of proposed Uniform Assessment Laws which will be submitted to the various State Legislatures for consideration and adoption. The State, through the Legislature, subject to constitutional limitations, has authorized these quasi-municipal corporations of limited powers to act as its agents. The customary legal liability of the State as principal for its agent's authorized acts in issuing securities does not result in the State becoming liable for the debts of these corporations. The State Constitutions limit the powers of municipal corporations in the matter of incurring debt, and, to a greater degree, the indebtedness of district corporations. The issue of securities to pay for a present debt, such as a warrant represents, differs greatly from the future pledging of district credit. A warrant promises to pay when funds are available, while a bond is a future promise to pay for funds presently loaned by the purchaser.[11]

The cost of designed roads must be financed from funds secured by the sale of district securities to the investor. The engineer must consult with his attorney and his district's banker before he can be assured of financing his projects. He must have an understanding of the corporate financial powers and the sale of its securities to the same extent that he has materials tested and traffic conditions analyzed.

In conclusion, it may be stated that the estimated annual revenues from motor vehicle users in the nature of automobile licenses and gasoline tax will prove most valuable as collateral assurance that the original sources of financing construction through municipal corporate securities will be performed. These license taxes are operating revenues and should be distinguished from highway district bonds, which are in the nature of first mortgage liens on benefited real estate. The highway engineer must not ignore the funds from bond sales, which make the original cost of construction possible. It is the finished structure that creates the "readiness to serve" the public (mentioned in Mr. Crum's paper), which is productive of annual revenues from motor vehicles. The owners of the latter, in turn, pay the State license and gasoline tax.

[11] Dillon, "Municipal Corporations," Chapters XIX and XX.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN OF LARGE PIPE LINES

Discussion

BY MESSRS. C. P. VETTER, L. E. GRINTER, AND OREN REED

C. P. VETTER,[48] ASSOC. M. AM. SOC. C. E. (by letter).[48a]—The author has made a valuable contribution to the art of designing large pipe lines subject to low pressures. The manner of support advocated by him is not entirely new, and several attempts have been made by others to produce a satisfactory stress analysis of which at least one is entirely rational, although, unfortunately, far too cumbersome for practical use. This is the analysis by Mr. A. Frey Samsioe mentioned by the author under "Case 3." Mr. Schorer deserves credit, therefore, for the suggestion of applying the membrane theory (developed largely for the design of domes and vaulted roofs) to the problem, and thereby establishing surprisingly simple formulas.

Some time may elapse before designers gain confidence in the author's membrane equations, as has been the case with every innovation in engineering mathematics. Mr. Schorer would undoubtedly help his cause by giving details of some of the tests that have been conducted under his auspices. There is no doubt that his solution is an approximation, good only for thin pipe shells. Its value, however, may best be determined by tests since the exact mathematical solution is so complicated that it is practically unavailable.

Unfortunately, no details are given of the derivation of the primary equations, (1), (2), and (3), and Fig. 2 is of little help because the all-important increments of stress which form the basis for the stress equations, have been omitted. Therefore, an attempt at a rational stress analysis along the lines suggested by the author may be of interest.

Fig. 35 shows a section of pipe shell subjected to a radial load, p, in pounds per square inch, and a mass load (weight), w, in pounds per cubic

NOTE.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in September, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1931, by Messrs. L. J. Mensch and W. P. Roop; December, 1931, by Messrs. Johannes Skytte, Donald E. Larson, Raymond J. Roark, and F. W. Hanna; January, 1932, by Messrs. Paul Bauman and L. J. Larson; February, 1932, by Messrs. C. M. Orr and Edward J. Bednarski; and April, 1932, by Messrs. H. C. Boardman, R. L. Templin and R. G. Sturm, and F. Knapp.

[48] Asst. Civ. Engr., Pacific Gas & Elec. Co., San Francisco, Calif.

[48a] Received by the Secretary February 15, 1932.

inch. The pipe shell is assumed to possess that special quality by which it is capable of sustaining normal forces and shear, but not bending moments. The section is part of a circular cylinder with its axis along the X-axis (in the diagram). In so far as its incapacity to resist bending moments is concerned the shell represents, in space, something similar to a string or a cable in a plane. The principal difference is that a string cannot sustain internal shear while the pipe shell can. If it is assumed that the shape of

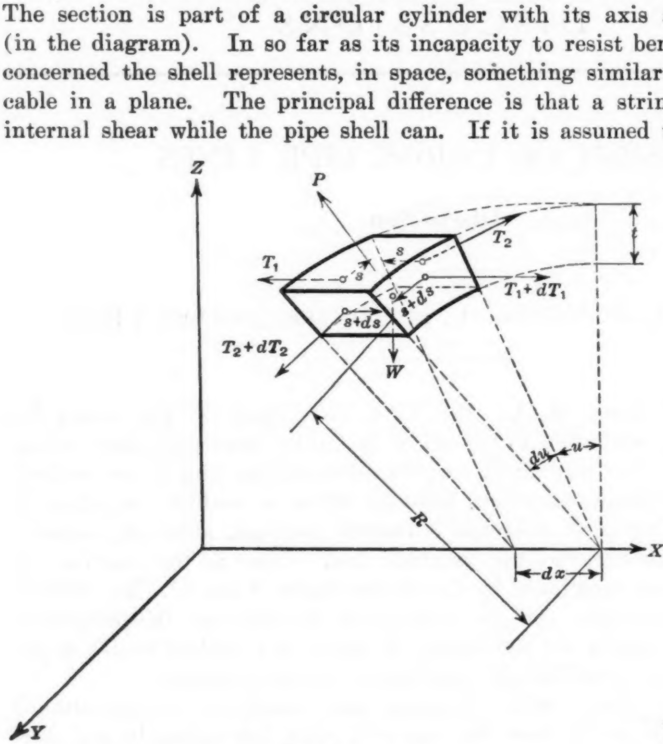

FIG. 35.

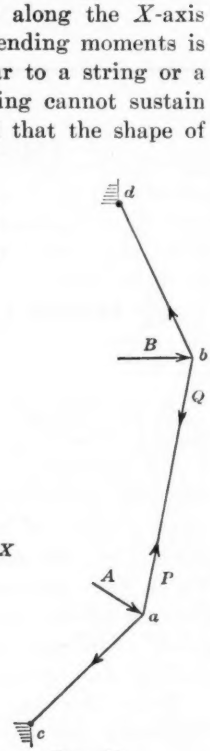

FIG. 36.

the string under load were known by intuition, and the corresponding internal stresses were found by applying the principles that shear and bending moments must not occur, then the assumed shape is a true one if each and every point of the string is found to be in equilibrium.

Fig. 36 will illustrate this reasoning: Let A and B be two forces acting on a string of the assumed shape, *c-a-b-d*. Under the assumptions that shear and bending cannot exist, the internal stresses, p and q, are found. If p is equal and opposite to q, then every point of the string is in equilibrium and the assumed shape of the string is stable.

Applying this principle to the pipe shell of which a section is shown in Fig. 35, the outside forces are p and w; the assumed shape is a circular cylinder; and the problem is to find the internal stresses under the assumption that bending moments cannot occur. If, after the internal stresses have been determined, it an be proved that every point of the shell is in equilibrium, then the assumed circular cylinder shape is correct and the structure is stable.

The author starts out, as do the originators (Messrs. Kraus and Dischinger) of the membrane formulas, by assuming an outside force with

components, X, Y, and Z. If the outside force is to be taken as a surface load this is not possible, because any component in a tangential plane would necessarily produce both radial shearing stresses and bending moments in the shell. Surface loads, therefore, must be confined to radial loads, that is, loads perpendicular to the surface. Mass loads, such as gravity, are not surface loads, and are not affected by this rule. They may have any direction and, in Fig 35, are assumed parallel to the Z-axis (or the Z-axis is assumed vertical).

Returning to Fig. 35, the influence of p only is considered. Assuming $w = 0$ and by projection on the X-axis:

$$(T_1 + dT_1)\,R\,t\,du - T_1\,R\,t\,du + (s + ds)\,t\,dx - s\,t\,dx = 0$$

and,

$$dT_1 = -\frac{1}{R}\frac{ds}{du}\,dx \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (187)$$

and by projection on p:

$$p\,R\,du\,dx = T_2\,t\,dx\,\frac{1}{2}\,du + (T_2 + dT_2)\,t\,dx\,\frac{1}{2}\,du$$

and,

$$T_2 = \frac{pR}{t} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (188)$$

and, finally, by projection on an axis perpendicular to p and T_1:

$$(T_2 + dT_2)\,t\,dx - T_2\,t\,dx + (s + ds)\,t\,R\,du - s\,t\,R\,du = 0$$

and,

$$ds = -\frac{1}{R}\,\frac{dT_2}{du}\,dx \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (189)$$

Equations (187), (188), and (189) are the three primary differential equations determining the stresses. The case investigated here deals with a pipe just full of a fluid with unit weight, q. Therefore:

$$p = Rq\,(1 - \cos u) \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (190)$$

and by inserting in Equation (188):

$$T_2 = \frac{q\,R^2\,(1 - \cos u)}{t} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (191)$$

and by differentiation: $\dfrac{dT^2}{du} = \dfrac{qR^2}{t}\sin u$; which inserted in Equation (189) gives,

$$ds = -\frac{qR}{t}\sin u\,dx \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (192)$$

By integration with respect to x: $s = -\dfrac{qR}{t}\,x\sin u + f_1(u)$ and, again, by differentiation with respect to u: $\dfrac{ds}{du} = \dfrac{qR}{t}\,x\cos u + \dfrac{d\,[f_1(u)]}{du}$.

Inserting in Equation (187):

$$dT_1 = \frac{q}{t}\, x \cos u\, dx - \frac{1}{R}\, \frac{d\,[f_1(u)]}{du}\, dx$$

and by integration with respect to x:

$$T_1 = \frac{1}{2}\, x^2\, \frac{q}{t} \cos u - \frac{1}{R}\, x\, \frac{d\,[f_1(u)]}{du} + f_2\,(u) \dots\dots\dots (193)$$

in which, $f_1\,(u)$ and $f_2\,(u)$ are the integration constants. They represent functions that may contain the variable, u, but not x, and their determination is not the least part of the problem.

Let it first be assumed that the pipe considered is the overhanging portion of a pipe line, such as would occur at an expansion joint. Then, by choosing the origin of the co-ordinate system at the end of the pipe, it is evident that for $x = 0$, $T_1 = 0$, and $s = 0$. Therefore, $f_1\,(u) = 0$ and $f_2\,(u) = 0$, and, consequently:

$$s = -\frac{qR}{t}\, x \sin u = -\frac{q}{t}\, x\, y \dots\dots\dots\dots\dots\dots (194)$$

and,

$$T_1 = \frac{1}{2}\, \frac{q}{t}\, x^2 \cos u = \frac{1}{2}\, \frac{q}{tR}\, x^2\, z \dots\dots\dots\dots\dots\dots (195)$$

Equation (195) shows that for constant x, the direct stress is proportionate to the Z-co-ordinate, indicating that except for deflections due to shear, the cross-section remains plane under load, assuming proportionality between stress and strain. The stress, T_1, may be expressed, therefore, by the well-known equation:

$$T_1 = \frac{M}{I}\, z \dots\dots\dots\dots\dots\dots\dots\dots (196)$$

which is based on this same assumption only and where M is the bending moment of the exterior forces and I, the moment of inertia of the cross-section. The moment is expressed by $M = \frac{1}{2}\, \pi\, R^2\, x^2\, q$, and,

$$I = 4 \int_0^{\frac{\pi}{2}} R^3\, t \cos^2 u\, du = \pi\, R^3\, t$$

Therefore, $T_1 = \dfrac{\frac{1}{2}\pi R^2 x^2 q}{\pi R^3 t}\, z = \dfrac{1}{2}\, \dfrac{q\, x^2}{tR}\, z$, which is identical to Equation (195). However, the analogy goes still further. If the cross-sections remain plane, the shearing stresses may be expressed by another well-known equation: $s_z = \dfrac{M_z}{I}\, s_v$, in which, s_z is the total horizontal shear per unit length in two radial sections having a distance, z, from the neutral axis; M_z is the static moment with regard to the neutral axis of that part of the cross-section which lies outside the radial sections; and s_v is the total vertical shear. The moment is expressed by:

$$M_z = 2 \int_0^u t\, R^2 \cos u\, du = 2\, t\, R^2 \sin u$$

Furthermore, $s_v = \pi\, R^2\, x\, q$, and, therefore, $s_z = 2\, p\, x\, R \sin u$. The unit shear equals $s = \dfrac{\frac{1}{2} s_z}{t} = \dfrac{q R}{t}\, x \sin u$ which numerically is identical to the value given by Equation (194).

In a pipe supported at either end the origin of the co-ordinate system may be chosen at the section where the total vertical shear, s_v, is zero. For a pipe with uniform load, such as investigated herein, the origin would be in the middle between the supports, as suggested by the author.

From Equation (192) for $x = 0$,

$$s = f_1\,(u) \qquad \text{(197)}$$

and,

$$s_v = 2 \int_0^{\pi} s\, R\, t \sin u\, du = 0 \qquad \text{(198)}$$

By inserting Equation (197) in Equation (198) and eliminating the constant values:

$$\int_0^{\pi} f'\,(u) \sin u\, du = 0 \qquad \text{(199)}$$

This equation is satisfied by:

$$f'\,(u) = 0 \qquad \text{(200)}$$

and also by numerous other trigonometrical functions. It is evident, however, that these other functions would not agree with the law of continuity, at least not for uniformly distributed load. It must be assumed, therefore, that Equation (200) gives the only possible solution of Equation (199).

Equation (192) then becomes:

$$s = -\frac{q R}{t}\, x \sin u \qquad \text{(201)}$$

as for cantilevers, Equation (193) becomes:

$$T_1 = \frac{1}{2}\, x^2\, \frac{q}{t} \cos u + f_2\,(u) \qquad \text{(202)}$$

for $x = \dfrac{L}{2}$, $T_1 = 0$, in which, L is the length between supports, therefore:

$$f_2\,(u) = -\frac{1}{8}\, L^2\, \frac{q}{t} \cos u \qquad \text{(203)}$$

and,

$$T_1 = -\frac{q}{8\,t}\,[L^2 - 4\,x^2] \cos u$$

or,

$$T_1 = -\frac{q}{8\,t R}\,[L^2 - 4\,x^2]\, z \qquad \text{(204)}$$

By comparing Equation (204) with Equation (195) it is seen that in the case of a pipe supported at either end, the normal stresses are also proportionate to the Z-co-ordinate. It is evident, therefore, that the stresses are the same as those found by the common beam formulas.

This point is of particular interest because in the ordinary beam theory it is necessary to assume that a normal section remains plane under stress in order to establish the usual formulas for the stresses. In the case of thin shells this assumption is not necessary. Neither is it necessary to assume proportionality between stress and strain nor a constant elastic modulus. If, on the other hand, this proportionality exists, the cross-sections will necessarily remain plane.

Thus, it has been shown that the ordinary bending formulas may be applied to this type of cylindrical shells in the case of cantilevers and simple beams. The reasoning may easily be extended to show that they apply also

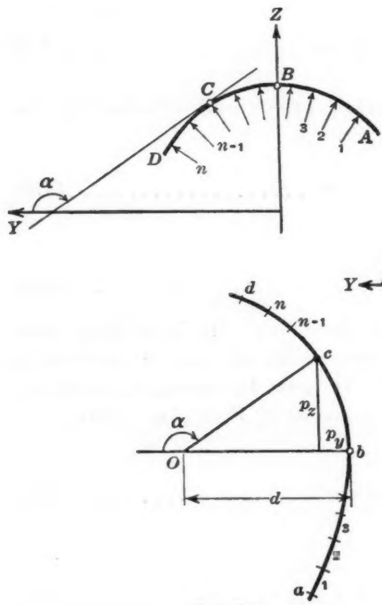

FIG. 37.

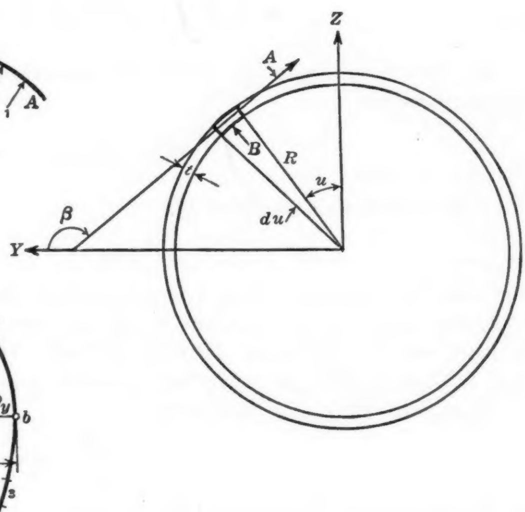

FIG. 38.

in the case of continuous beams. In addition to the bending stresses, hoop stresses occur, which are determined by Equation (191). They are not affected by the manner in which the pipe is supported.

It remains to be proved that an arbitrarily chosen element of the shell is in equilibrim when subject to the forces and stresses developed in the preceding equations. A circular ring, cut from the pipe, and of unit length, may be chosen for the proof. It is well known that a circular ring is in equilibrium if the string polygon to all the forces acting upon it is a circle. The equation for the string polygon may be found as follows: Let 1 to n (Fig. 37(a)) be the external forces, which are infinitely small, infinitely close

together, and which form a continuous load. Let $A\ B\ C\ D$ be a string polygon to these forces, B being the point at which the string polygon is parallel to the Y-axis, and C being an arbitrary point with co-ordinates (y, z). The force polygon is shown in Fig. 37(*b*). The points, *a, b, c,* and *d,* correspond to A, B, C, and D, and O is the pole. Then, the differential equation for the string polygon may be expressed by $\frac{dz}{dy} = \tan\alpha = -\frac{p_z}{d - p_y}$, in which, d is the pole distance. The equation is obviously correct irrespective of the magnitude of d. If $d = 0$, the string polygon becomes the pressure line to the forces, 1 to n, the equation for which becomes:

$$\frac{dz}{dy} = \frac{p_z}{p_y} \dots\dots\dots\dots\dots\dots\dots\dots (205)$$

in which, p_z and p_y are, respectively, the sum of the Z-components and the sum of the Y-components of all the forces between B and C.

Returning again to the circular element of the pipe (Fig. 38), the external forces are: A, the increment in shearing stress for the length l, of the element, and B, the water pressure. The value of A may be obtained from Equation (192) as follows:

$$A = t\,R\,du\,\frac{ds}{dx} = +\,q\,R^2 \sin u\,du$$

The sign is a plus because the positive direction of the increment, as shown in Fig. 35, has been reversed. The value of B may be obtained from Equation (190), as follows:

$$B = p\,R\,du = q\,R^2\,(1 - \cos u)\,du$$

Then, $p_z = \int_0^u [A \sin u + B \cos u]$, or,

$$p_z = R^2\,q\,(\sin u - \frac{1}{2} \sin 2u) \dots\dots\dots\dots\dots\dots (206)$$

and, similarly, $p_y = \int_0^u [B \sin u - A \cos u]$, or,

$$p_y = R^2\,q\,[\frac{1}{2}\,(1 + \cos 2u) - \cos u] \dots\dots\dots\dots (207)$$

and,

$$\frac{p_z}{p_y} = \frac{\sin u - \frac{1}{2} \sin 2u}{\frac{1}{2}\,(1 + \cos 2u) - \cos u} = -\tan u = \tan\beta \dots\dots\dots (208)$$

and, $\frac{dz}{dy} = \frac{p_z}{p_y}\ \tan\beta$.

This is readily seen to be the differential equation for a circle with the center at the origin of the co-ordinate system.

It has thus been proved that the pressure line for the forces acting on a circular ring cut from the pipe is itself a circle. Therefore, the circular shape is stable, and the assumption that no bending moments would occur is correct.

If the same reasoning were applied to the case of dead load without water pressure, it would be found that the author's equations throughout rest on a solid foundation of logic. It is hoped that this detailed discussion of the basic part of the author's paper will help it to get the recognition it so highly deserves.

L. E. GRINTER,[49] ASSOC. M. AM. SOC. C. E. (by letter).[49a]—In spite of its brevity, this paper has real value. If it succeeds in calling the attention of designers to the serious stresses that exist in ring stiffeners and in the shell near the supports, the author will have performed a valuable service. Observation shows that the use of inadequate ring stiffeners has been a common mistake.

At the beginning of this paper Mr. Schorer states that the theory developed is applicable to horizontal tanks as well as to pipe lines. He then advocates the use of ring stiffeners at supports only. It is important to realize that horizontal tanks exceeding 12 ft. in diameter are used in industrial plants where the shell may not be exposed to internal pressure. Since the most serious condition exists with the shell full or partly full of liquid, it is clear that the thickness required to resist direct ring stresses is exceedingly small.

Storage tanks of this type might easily be damaged during erection or while in service so that the cross-section would be far from circular. The writer considers the use of thin plates, stiffened at reasonable intervals by intermediate ring stiffeners, to be the proper design in such cases. Wherever resistance to corrosion or internal pressure requires reasonably thick plates, the intermediate stiffeners may be omitted. The important consideration is to produce a structure of such stiffness that any possible change in shape will not produce leakage at the joints.

The safety of a large pipe line or storage tank in which intermediate stiffeners are not used, is dependent largely upon the proper design of ring stiffeners at the supports. The computation of bending stress in the ring is discussed by the author for a special case. The general problem may be solved by any one of several methods, but the "column analogy" method[50] introduced by Hardy Cross, M. Am. Soc. C. E., seems to have the advantage of simplicity. With the aid of Mr. Schorer's equations for shear, which control the load on the stiffener ring, the column analogy may be used to solve directly for the moments irrespective of the type of support.

The writer is fundamentally opposed to the common procedure of analyzing special cases and recording the results as formulas for use in practice. For instance, Mr. Schorer's equations for moments in the ring stiffener are developed for a special type of support, namely, a lug support at each end of the horizontal diameter. On the other hand, the structures which the author lists in his "Synopsis" are quite as likely to be supported by a cradle or by some other special device. The designer would find considerable difficulty in revising these formulas to fit other types of support.

[49] Prof. of Structural Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[49a] Received by the Secretary February 23. 1932.

[50] "The Column Analogy," by Hardy Cross, M. Am. Soc. C. E., *Bulletin No. 215*, Univ. of Illinois Eng. Experiment Station.

Another criticism inherent with formularization is that the simplified appearance of the final results leads poorly trained engineers to attempt their use. The outcome is likely to be unsatisfactory to all concerned since such a designer may have almost no conception of the important change in stresses which frequently accompanies an apparently slight change in the type of support. The use of a general method of analysis where numerical computations are carried through the entire process tends to eliminate such careless design work. The column analogy method, therefore, may be considered as a tool for extending the usefulness of the author's work.

Many of Mr. Schorer's equations are taken from the mathematical theory of elasticity. Unfortunately, it is not possible to suggest a general method of analysis to take the place of such formulas. Most technical designers do not have the mathematical training which is required to make use of the mathematical theory of elasticity, and, therefore, they must depend upon the results of the work of specialists in this field. The writer believes that a large amount of poor design work is inevitable unless the analysis of all such problems is simplified until it can be understood by the designer.

Mr. Schorer's equations for stresses in the shell near stiffener rings are of great importance. Such flexural stresses would be overlooked by many designers, although they could scarcely miss the stresses caused by longitudinal beam action. The author's numerical example is well chosen since it points out the importance of the rim stress by showing it to be of much greater magnitude than the longitudinal beam stress despite the fact that the unsupported length is 60 ft.

OREN REED,[51] ASSOC. M. AM. SOC. C. E. (by letter).[51a]—Ring supports for large pipe lines have several advantages over the usual broad concrete saddles. Generally, no attempt is made to determine the stresses in the pipe shell near the supports with the old type saddle. This is not necessary with a pipe having a heavy shell, but a knowledge of the stress distribution and the possibility of distortion of the pipe over the supports are essential, when a pipe of large diameter and thin shell is to be used. Computation of the stresses in the pipe shell over a saddle support is complicated and uncertain, while with ring supports the stress distribution can be computed more readily.

Ring supports probably should not be used on a pipe line where there is a possibility of collapse due to vacuum. In the event of even a partial collapse, joints might be pulled apart in such a way that a reconditioning of the line would be difficult. However, air-valves and vents should always be provided at summits in the line.

In 1923, the writer inspected the Svaelgfos No. 2 penstock, in Norway. This pipe line is about 7 ft. in diameter and has heavy stiffener angles at about 6-ft. intervals. As pointed out by the author, such a pipe line does not have a pleasing appearance due to the use of the stiffeners.

If the pipe sections are to be transported a long distance, especially by boat, or handled several times before they reach the point of installation, it

[51] Field, Engr., Indiana Dept. of Conservation, Indianapolis, Ind.

[51a] Received by the Secretary March 1, 1932.

may be necessary to use stiffening angles for the thin plate sections, unless a definite minimum thickness is adopted. Such a minimum thickness is often much heavier than would be required to withstand the static and dynamic pressures at the location in the line.

At most Swedish hydro-electric plants, heads are not great and the fall is concentrated so that an open penstock is seldom used. However, there are two recent examples of the use of ring supports for pipe lines, at Atrafors and at Tanger. The Atrafors pipe has a diameter of 13.1 ft., a plate thickness of $\frac{7}{32}$ to $\frac{9}{32}$ in., and the span between supports is 44.2 ft. The Tanger power plant was completed in 1931. The penstock has a diameter of 9.2 ft. and operates under a static head of 98.5 ft. above the bottom of the pipe. The line is 2 625 ft. long, and the normal span is 40 ft. between supports. The supporting rings are built up of plates and angles, and rocker-bearings transmit the load to small concrete piers on each side of the pipe. The use of such rocker-bearings makes the design of anchors less difficult. The connection point of the bearings makes an angle of about 30° with the horizontal center line. Ring supports were adopted in order to keep the project construction cost as low as possible.

Ring supports were also used on the Rempen penstock, of the Waggital Project, in Switzerland, which was completed in 1924. The two penstocks have a diameter of 7.2 ft., a plate thickness of $\frac{3}{4}$ in., and the span between supports is 73.5 ft.

The author will have done the Engineering Profession a worthwhile service, if by his design of the Glines Canyon penstock, and the detailed description, a more general use of ring supports for large pipe lines is adopted in America.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

SOIL MECHANICS RESEARCH

Discussion

BY MESSRS. GEORGE PAASWELL, JOHN H. GRIFFITH, AND H. E. GRUNER

GEORGE PAASWELL,[96] M. AM. SOC. C. E. (by letter).[96a]—This paper gives a sober account of the work done under the direction of Professor Gilboy, and it is primarily educational.

There can no longer be any doubt that the rule-of-thumb era in foundation work has passed. Foundation failures can no longer be ascribed to the perverse workings of a balky Nature. The proper design of a foundation is not simply a surface study of the loaded area. Bearing in mind that the loaded surface of the soil creates a certain stress distribution through a depth, comparable in extent with the area of surface loaded, it can easily be seen that an extensive soil exploration is required before a proper anticipation may be made of the distortion of the soil under load. Surface tests form merely one link, then, in a complete study of the foundation.

The vast amount of sub-surface construction in large cities has created a field for an extensive study of foundation problems. Not only must the foundation selected be of such character as to sustain safely the load to be placed upon it, but also the method of constructing this foundation must be such as not to endanger existing neighboring foundations. This has required a type of research that finds its conclusions from the applications of soil mechanics as given in the paper.

The writer was privileged to collaborate with both Professors Terzaghi and Gilboy on an extensive series of experiments in connection with the construction of the Houston Street Subway in New York City, mentioned by the author. On this work the sand extended in a deep layer down to rock which varied 60 ft. to more than 100 ft. below the ground surface. The sand

NOTE.—The paper by Glennon Gilboy, Jun. Am. Soc. C. E., was presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929, and published in October, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: December, 1931, by Messrs. J. C. Meem and H. de B. Parsons; January, 1932, by Messrs. Jacob Feld and John R. Jahn; February, 1932, by Messrs. C. H. Eiffert, A. A. Eremin, F. N. Menefee, and E. G. Walker; and April, 1932, by Messrs. D. P. Krynine, J. Stuart Crandall, and Frank Alwyn Marston.

[96] Engr., Corson Constr. Corporation, Brooklyn, N. Y.

[96a] Received by the Secretary February 3, 1932.

varied in texture from extremely coarse to extremely fine, distributed in irregular layers in so far as its texture was concerned. Its density bore no relation to the depth at which the density was ascertained so that the rule-of-thumb generally applied, which gives the bearing capacity as increasing with the depth, was entirely in error. The angle of internal friction as found by experiments on small samples of the sand was fairly constant, a fact which the writer believes has been justified by some later experiments conducted by Professor Gilboy, which show only small variations in the angle of internal friction in varying densities of sand. It was also found that for the small loads applied (none exceeded 5 tons per sq. ft.) the settlement was not a function of the area loaded.

This is an important conclusion, although it is one that has been frequently checked by experiment in sands, and it must be emphasized that it probably applies only to moderate loads. For very heavy loads, such as are carried by pile clusters with concentrated point loads, experiments seem to show that the settlement is a function of the area loaded. The writer hopes that this important topic will be a subject of future study and experiment, since it has been found that in clays the settlement is intimately related to the shape and area of the surface loaded.

The effect of surcharge was studied carefully in these experiments, and the settlement was noticeably decreased by the presence of a surcharge of sand around the loaded area; that is, the surface loading settles more rapidly under a given load than under the same load applied on the same area at a distance below the surface of the ground. When the load is applied at a depth below the surface equal approximately to the maximum diameter of the loaded area, the maximum effect of surcharge is obtained. This fact should be carefully noted in connection with the underpinning of buildings adjacent to a future excavation. The piers sustaining the existing building should be carried to a depth equal at least to the thickness of the pier below the depth of the future excavation.

The sum total result of the experiments gave quite a comprehensive picture of the behavior of sand of this character within the range of loads to be placed upon it, and was sufficiently substantiated by other observations and by later behavior of the finished structure to vindicate the time and money spent on these experiments and to give striking proof that soil mechanics was able to answer decisively the questions of practical and immediate importance.

The design of a system of sheeting and bracing of unique character was governed primarily by the observations taken on the retaining-wall experiments quoted by the author. The various phases of movement in a retained bank of sand behind a yielding support (such as sheeting), was checked by observations during the construction of the subway. In some cases, the deflections of the sheeting were quite considerable, but no fracture occurred, showing the diminution in pressure as the retaining wall moved away from the applied load. It need hardly be mentioned here that a system of sheeting and bracing to sustain safely a deep bank of sand must be most carefully designed if the results of the quoted experiments are to be applied in a safe manner. Vast economies in subway construction have resulted to the City

of New York because of the successful application of this type of sheeting and bracing, and it would not be "far fetched" to trace back this vast saving of money to the city to the series of experiments, starting in Constantinople, Turkey, and ending in Boston, Mass., as conducted in that brilliant series of studies made by Professor Terzaghi.

JOHN H. GRIFFITH,[97] M. AM. SOC. C. E. (by letter).[97a]—In regard to the question of stress distribution, Professor Gilboy states (see "Foundations"):

"* * * while the classical theory of Boussinesq is the best available, it can yield only approximate results. The analysis of the stress-strain characteristics of soils thus far performed includes only the simplest cases. Most important of all, some assumption as to homogeneity of the underground must be made, and this is only approximately fulfilled, even under the most favorable circumstances."

Following this is the apparently contradictory statement that such analyses yield valuable information. He cites as proof the instance of a highly heterogeneous matter consisting of piles overlying an area of mud 120 ft. below the surface, and comments, as follows: "The results agreed remarkably well with the observed values, showing that such studies are of considerable practical importance, * * *."

To understand the disparities of opinions which have originated with respect to earth analyses, or to use Professor Gilboy's term, "soils," it is well to consider that the theory of earth resistance has developed two schools; those who, like Professor Gilboy, take the physicist's, or molecular, point of view involving grain parameters, shapes of particles, true and apparent cohesions, intrinsic stresses, and structures of atoms; and those who adhere to the viewpoints of mathematicians, that such conceptions have no place in the systematic working formulations of science. It is important to consider the facts in this case.

Strictly speaking there are no systematic atomic or molecular theories formulated as such. They are unmanageable in science and much more so in applied physics, such as engineering and geology. A true molecular analysis is a problem in *n* body dynamics, with *n* very large, and involves a complex theory of numbers or a calculus of finite differences which the proponents of such methods do not seem to have at their command.

On the contrary the systematic theories of science involve molar matters the properties of which are defined by continuous functions except for discontinuities at particular points and boundaries. One may consider as examples among others the theories of electro-magnetism, thermodynamics, elasticity, hydrodynamics, meteorology, seismology, and relativity. They are fundamentally field theories. The field is the measurable entity, the only thing that can be measured. If an oscillator vibrates in mercury it immediately diffuses its effect into waves as in the case of a tuning fork. The tuning fork is a singularity or peculiarity of the wave field, the point or region where the lines of force converge, and the function may increase without limit.

[97] Prof. of Experimental Eng., Iowa State Coll., Ames, Iowa.

[97a] Received by the Secretary February 8, 1932.

Having the equation defining the field it is necessary to find the peculiarities the function may assume in limiting cases and at points and special boundaries.[98] In seismology the engineer seeks for the focus of disturbance, in electro-magnetism for the pole or electrode, in acoustics for the source, and their functional aspects, together with the types of existence theorems. The same type of problem arises in all branches of dynamics, for the different modes of energy.

The potential theory formulated for matter by George Green,[99] while strictly a molar theory, is really applicable to molecular systems composed of an immense number of mutually re-acting particles, as stated by its author, and without the necessity of multitudinous borings, chemical and colloidal analyses, as the writer will proceed to show.

Several investigators have made tests[100] showing that the vertical distribution of pressure under a load appears to be in agreement with the mathematical distributions formulated by Boussinesq[101] and Michel.[102] The writer has suggested the availability of Boussinesq's formula,

$$Z_z = \frac{3}{2\pi}\frac{P z^3}{r^5} \quad \text{......................} (20)$$

treating the coefficient as an empirical constant in the usual manner illustrated in hydraulic formulas for orifices, which is entirely independent of physical constants, involving only the load, P, and the location of stress at the point distant r from P.

Anson Marston,[103] Past-President, Am. Soc. C. E., applied Boussinesq's theory to the problem of bridge rollers in 1894, and supplemented this by researches on large wheels in later years. In the Progress Report on Culvert Pipe Investigations covering 1915 to 1921, submitted to the U. S. Bureau of Public Roads and the various State Highway Departments, he showed by tests that Boussinesq's formula for a concentration (Equation (20)) applies to all levels of fill to within 2 ft. of the load "source" where the full concentration applies; the analytic function becomes infinite with the radius vector measured from the load reducing to zero. A little later, Messrs. Spangler, Mason, and Winfrey[104] demonstrated that the formula gave the upper limit of load and generally showed a close parallelism with Boussinesq's law. In 1920, at a lecture before a group of railroad officials, Dean Marston stated that the lines of force crowd into the culvert or pipe tending to break it, the weight sustained being more than the vertical surcharge over the extrados.

In the most recent researches of Dean Marston[105] the theory of potential based on molar matter analyses, where the integrations have been carried

[98] "Physical Properties of Earths," *Bulletin No. 101*, Iowa Eng. Experiment Station, 1930.

[99] "Mathematical Papers," p. 243.

[100] *Bulletin*, Pennsylvania State Coll., Eng. Experiment Station, Vol. 17, p. 650; "The Distribution of Pressure by Granular Material", by M. L. Enger, M. Am. Soc. C. E., *Engineering*, Vol. 101, 1916, p. 170; *Proceedings*, Am. Soc. for Testing Materials, Vol. 17, 1917, p. 650; *Proceedings*, Am. Soc. C. E. August, 1920, Papers and Discussions, pp. 931–937; *Loc. cit.*, March, 1920, Papers and Discussions, pp. 251–304.

[101] "Theory of Elasticity", by E. A. H. Love, Formula, p. 189.

[102] *Proceedings*, London Mathematical Soc., Vol. 32, 1900, p. 23.

[103] *Transactions*, Am. Soc. C. E., Vol. XXXII (1894), p. 99.

[104] *Bulletin No. 79*, Iowa Eng. Experiment Station, pp. 14–15.

[105] *Bulletin No. 96*, Iowa Eng. Experiment Station, 1930, p. 17, Formula 13.

out for "concentrated super-loads" taken over the "conduit top horizontal plane projection," show excellent correlations according to a recent communication to the writer by Professor Robley Winfrey, of the Iowa Experiment Station. He has made a detailed study of the problem by comparing the results of summing Boussinesq's simpler formula under a concentration with the results obtained by integration and by dividing the stressed area into relatively small squares. The recent researches not only show that the potential formulas give the upper limits of the distributions, but actually apply when the load has close proximity with the extrados. As in the case of the simpler formula the results are independent of the moduli of the material applying to either gravel, sand, or clay, as long as the lattice structure of the medium is not too much deformed as would be the case in straining an ordinary bridge truss. If it experienced finite deflections, the panel shapes would change involving a suitable correction.

Researches of the kind mentioned are valuable epistemologically in showing inductively the sufficiency of the potential algorithm, as a complete physical method. One instance only will suffice to show the possibilities in the distribution of forces by railroad tracks.

The researches of Harold Medway Martin[106] show that the distribution over the track from the train load is a "witch-shaped" curve of the type originally deduced by Agnesi. Suppose a culvert is under the track, the stress may be obtained as given by the general formula (Equation (20)), with a load equal in amount to the ordinate of the track distribution of Martin, and summed or integrated for continuous distributions. In the case of the attractions of body upon body according to the Newtonian potential theory[107] the general potential of mass, m', with respect to the mass, m, is:

$$V = \int\int\int\int\int\int \frac{\rho\,\rho'\,dm\;dm'}{r} \quad \ldots\ldots\ldots\ldots(21)$$

in which, r is the distance between the mass elements of the bodies. Where there is an intervening medium, such as the earth matrix, it is necessary to introduce the direct and legarthmic potentials of Boussinesq in the manner shown in a previous paper.[108] In general, it is not possible to dispense with the elastic constants. This is the case in the instance of a retaining wall and for other elastic or semi-elastic distribution in which lateral pressures occur. The potential distributions hold very closely in the case of impact.[109]

The retaining wall problem is one of the oldest historically and is most difficult to formulate. The author's apparatus, designed with meticulous care, will doubtless throw new light on the problem. A solution by molecular dynamics will be of the greatest interest to the profession. In addition to his rational solution for the weir problem of hydraulics,[110] Boussinesq's theory of the retaining wall is perhaps the most important of his purely engineering

[106] "Statically Indeterminate Structures and the Principle of Least Work", rev. and reprinted from *Engineering* (Lond.), 1895, pp. 63 to 71.

[107] "Peirce's Newtonian Potential Function", Third Edition, rev. and enl., p. 42.

[108] "Theory of Elasticity", by E. A. H. Love, Second Edition, p. 189; and *Bulletin No. 101*, Iowa Eng. Experiment Station, pp. 31–35.

[109] *Bulletin No. 79*, Iowa Eng. Experiment Station, pp. 22, 34 *et seq.*

[110] "Hydraulics", by F. C. Lea, 1908, Edwin Arnold, Lond., pp. 104–108.

analyses. He devotes more than 170 pages of his treatise on the subject[111] to this important case treating a molecular problem throughout by a strictly molar theory. He shows that it is a more general case than the theory of elasticity, and he arranges his parameters so that as a limiting aspect the theory reduces to the elastic case as a limit.[112]

The hangers, V_1 and V_2, of the retaining wall apparatus (Fig. 13) will balance Boussinesq's wall friction which he takes into account, except for infinitely smooth walls. It will be impossible to measure the maximum overturning moment because this is exerted when no energy has been expended in frictions and small displacements, and the author's apparatus will give some indeterminate equilibria and settlings in manipulations of the "wall" part as a result of the movements. The three-faced holder for the earth will act as a clamp on the specimen; under the wall friction there will be considerable restraint, making it impossible for the "retaining wall" to experience the maximum thrust. It was suggested to the writer by Dr. O. H. Basquin, of Northwestern University, that dental rubber placed on a wall would eliminate the friction since this material is nearly incompressible and has only a small modulus of rigidity. His theoretical reasons are sound, but the writer has never tried the material in this manner. It may eliminate the frictions of the side walls in the present case.

In conclusion, the writer does not wish to be construed as asserting that atomic, molecular, colloidal, and crystalline researches are not valuable in themselves. On the contrary, they are of the highest scientific importance in developing correct philosophies of the nature of energy, ether, matter, and atomic and crystalline structures. The point emphasized in the discussion is that the systematic working theories of science are not molecular; they are and always have been essentially molar theories involving apparent continuous entities and transmission coefficients, and not discrete entities and grain parameters, except in isolated instances ultimately to be displaced as a wave theory displaces a discrete theory.

H. E. GRUNER,[113] M. AM. SOC. C. E. (by letter).[113a]—The interesting information contained in Professor Gilboy's paper prompts the writer to contribute a description of certain simplified methods of conducting similar experiments, so designed that the most important properties of soil can be measured right at the construction site.

After the general line of attack had been established by the pioneer work of Terzaghi and Krey, and testing apparatus had been developed at the Staatliche Versuchsanstalt für Wasserbau und Schiffbau, in Berlin, the writer undertook the construction of simplified devices for testing the soil used in the construction of the dam for the Albbruck-Dogern Power Station on the Rhine. The purpose of this research was twofold: First, to determine those soil constants required for calculations of the stability of the dam; and,

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[111] "Essai théorique sur l'Equilibre des Massifs Pulverulents", Bruxelles, 1876; "Application des Potentials", Paris, 1885.

[112] *Loc. cit.*, p. 61.

[113] Cons. Engr., Basle, Switzerland.

[113a] Received by the Secretary December 30, 1931.

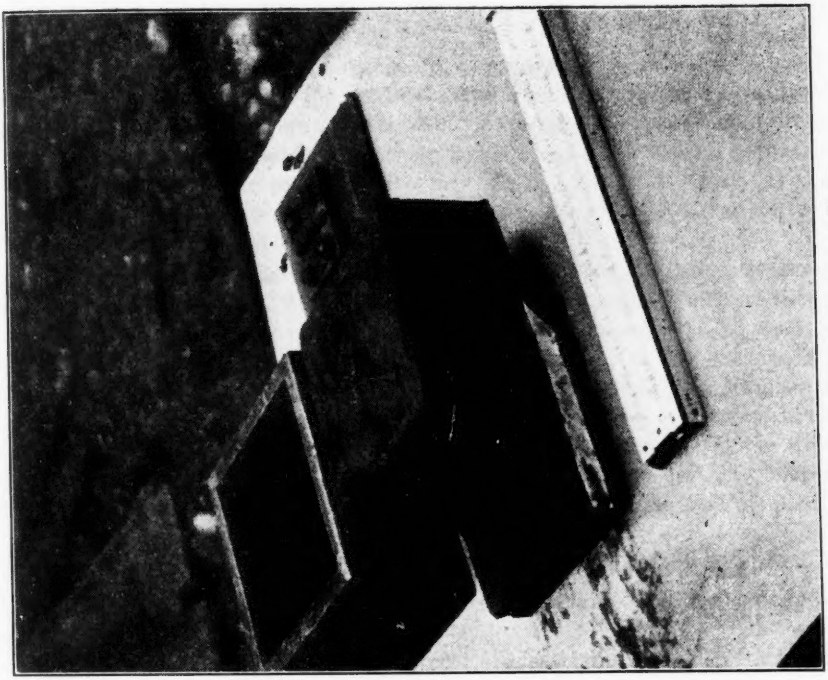

FIG. 41.—SAMPLE AFTER SHEAR TEST.

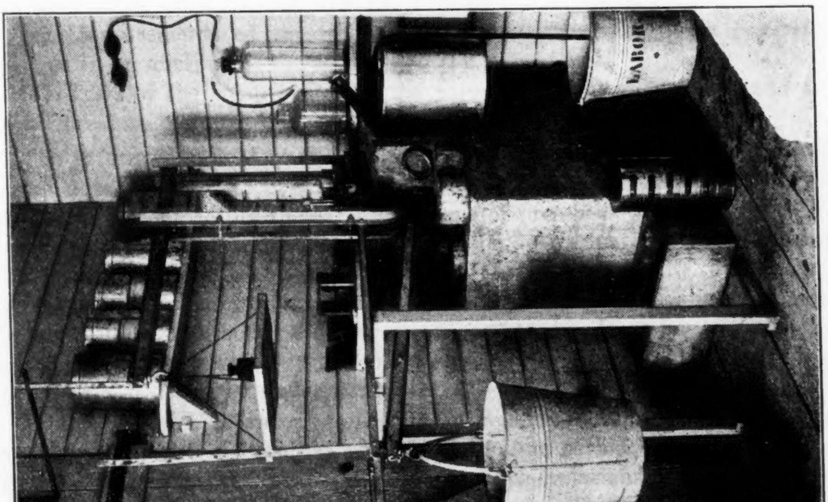

FIG. 40.—ARRANGEMENT OF APPARATUS.

second, to obtain continuous data on the properties of the material during the construction process.

On account of the wide variations in the properties of soils, it is necessary to have a definite basis for the selection of material suitable to a given purpose, and for the rejection of unsuitable material, such as soil containing a large proportion of colloids. It is the writer's opinion, therefore, that in the case of water-power projects involving large dams and canals a continuous series of soil tests is just as important as the continuous control tests customarily performed on cement and concrete.

Apparatus and Procedure.—A general view of the testing layout is shown in Fig. 40. Fig. 41 is a demonstration of the shear test, described subsequently. The devices are not intended to be as accurate as those used in research laboratories. They are well adapted, however, to the purpose for which they were designed, namely, to the rapid testing of a large number of samples, with a degree of accuracy sufficient for the analysis of dam construction materials. These instruments undoubtedly can be improved; in fact, the experiments to date have indicated the advisability of certain changes. The present description is intended merely to suggest one method of attacking the problem.

The tests thus far undertaken may be divided into five groups:

(1) Measurements of specific gravity, unit weight, and percentage of voids;

(2) Mechanical analyses, with the object of classifying the soil with regard to fineness;

(3) Permeability tests, to obtain data with which to determine the position of the line of saturation and the velocity of the seepage water;

(4) Measurements of shearing resistance, to determine those constants required in an analysis of the stability of the slopes; and,

(5) Tests of the compressibility, the variations in water content, and the lateral pressure of fine-grained materials.

The information in Group (5) is required for studies on the settlement of the dam, on the loss of water associated with consolidation, and on the possibility of compacting the earth by means of pressure.

Group (1).—These quantities enter into the determination of loading conditions in the dam. The specific gravity is determined by Schumann's volumenometer. The unit weight is obtained by the use of gauged capacity meters, different methods of filling being compared. In order to determine the change in unit weight produced by compression, the results of the consolidation test described under Group (5) are utilized.

Group (2).—There appear to be certain general relationships between the grading of a soil and its other properties, such as permeability and internal friction. However, the formulation of accurate relationships still requires considerable theoretical and experimental research. The grain distribution of sand is determined by sieve analysis. When dealing with mixtures of coarse and fine particles, all grains finer than 0.02 mm. are washed out, and the coarse material remaining is used in the sieve analysis.

The washing is performed with the aid of a cylindrical bottle, shown in Fig. 42(*b*). The length of time required for the operation can be computed from Stokes' formula. Using a depth of water of 20 cm. at a temperature of 15° cent., the time required for a particle, 0.02 mm. in diameter, to settle is about 10 min. Supplementary microscopic measurement of the largest grains showed that the formula gives very satisfactory results. However, the more accurate measurements made by Atterberg and Terzaghi showed that on account of the scale-like character of the particles the actual grain diameters are

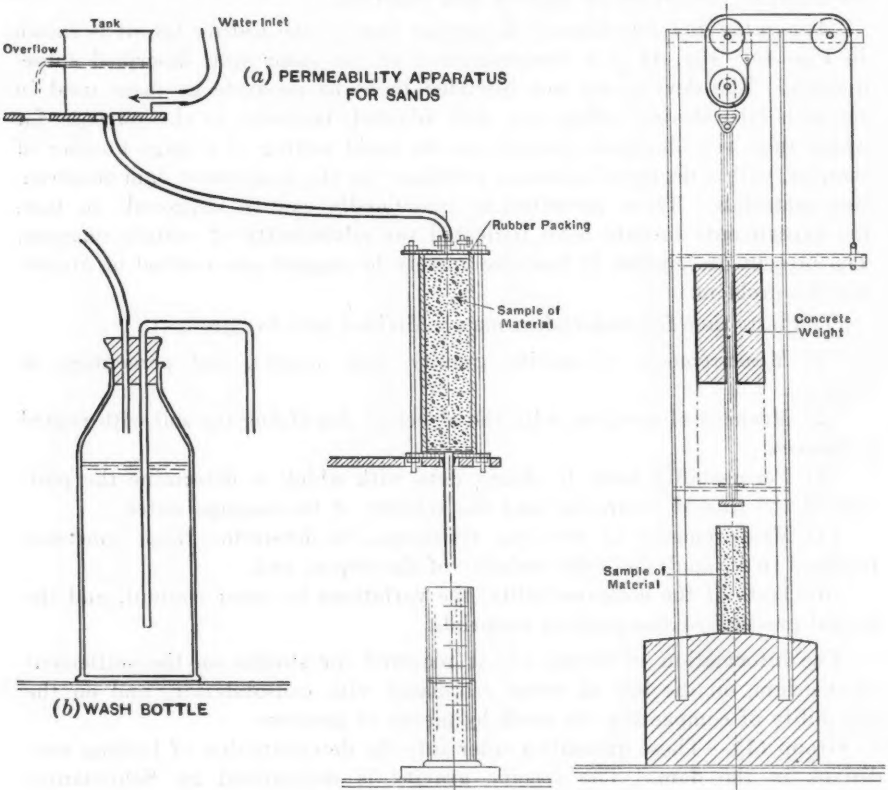

FIG. 42.

considerably larger, possibly twice the equivalent diameters. The equivalent diameter is defined as the diameter of a sphere which settles at the same rate as a flat grain of the same specific gravity.[114]

In determining the shearing resistance as a measure of the angle of internal friction, it is of especial importance to know the percentage of colloidal particles. Assuming that all particles smaller than 0.002 mm. may be classed as colloids, a sedimentation process similar to the foregoing may be used to

[114] "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., p. 40.

determine approximately the colloidal content. The settling time for a particle, 0.002 mm. in diameter, is about 16 hours. After this time has elapsed, the percentage of colloids is determined by optical measurements of the transparency of the turbid water.

Group (3).—The velocity of the seepage water is of particular importance in analyzing the safety of the dam against washouts. The permeability of sand is measured by means of a cylinder, obtained from the Versuchsanstalt für Wasserbau und Schiffbau in Berlin, and illustrated in Fig. 42(*a*). To ensure that the voids in the sample will be free of air, water is allowed to flow slowly into the cylinder from the bottom during the filling process, keeping the water level always slightly above the level of the sand. Thus, the sample is always completely immersed, and at the same time the washing out of very fine particles is avoided.

The influence of temperature on the coefficient of permeability may be determined from Justin's data.[115] In the test, measurements are first made on the value of the coefficient of permeability, k_0, which is defined as the apparent velocity of the water through the uncompressed soil under unit hydraulic gradient. Then, the sample is compressed under a pressure of 1 kg. per sq. cm. in the apparatus shown in Fig. 42(*c*), the change in volume is measured, and the permeability test is repeated on the compressed material, in order to determine the influence of the compression on the permeability.

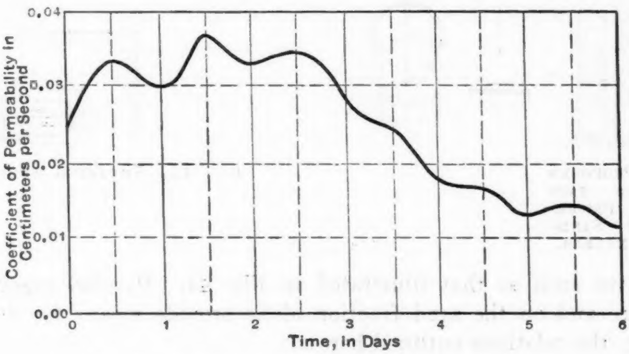

FIG. 43.—VARIATION OF PERMEABILITY WITH TIME.

The phenomenon of a partial sealing of the sample within itself during a period of continuous flow was investigated on several occasions. The permeability appeared to increase slightly during the first few days; but afterward it decreased rapidly and continuously, finally attaining a certain minimum value. A graph of this relation is shown in Fig. 43. In addition to the general tendency, it was observed that the permeability was subject to daily fluctuations of variable amplitude. The causes of these fluctuations could be studied only by performing accurate experiments in a scientific laboratory.

In order to determine the permeability of sand-gravel mixtures, the sand fraction, consisting of all grains smaller than 7 mm. in diameter, was sepa-

[115] "The Design of Earth Dams," by Joel D. Justin, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1.

rated from the sample by sifting; this fraction was then subjected to permeability tests as outlined herein. By making certain theoretical assumptions it is possible to determine the permeability of the mixture from the permeability of the sand fraction and the proportion of sand to gravel, provided the latter is such that all the voids in the gravel are completely filled with sand. However, the assumptions that must be made are open to question. In order to eliminate them, and to arrive at a correct result without the necessity of determining the proportions of the constituents of the mixture, it seems preferable to obtain the permeability of the mixture by direct measurement in

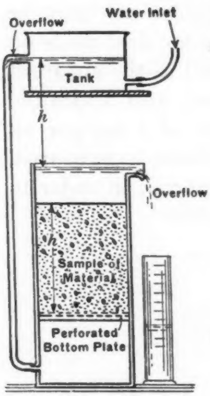

FIG. 44.—PROPOSED APPARATUS FOR MEASURING PERMEABILITY OF SAND-GRAVEL MIXTURES.

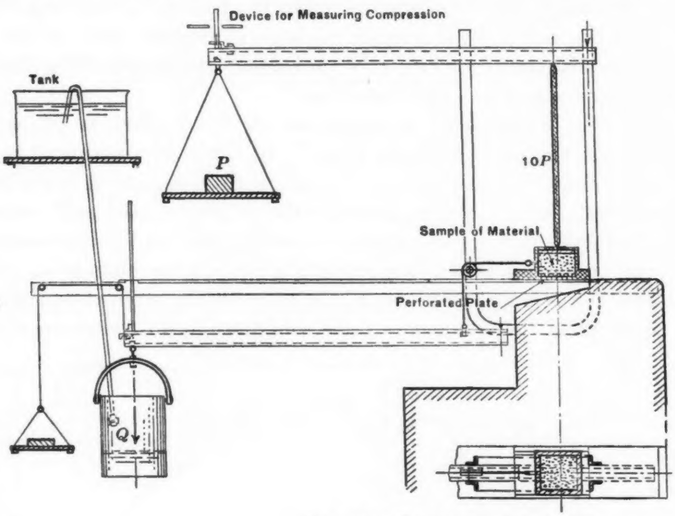

FIG. 45.—SHEARING APPARATUS.

an apparatus such as that illustrated in Fig. 44. Parallel experiments on the mixture and on the sand fraction alone provide means for determining empirically the relations outlined herein.

Group (4).—The shearing resistance of fine-grained soils, as a measure of the angle of internal friction, is determined by means of the apparatus shown in Fig. 45. Fig. 41 shows the appearance of a sample after it has been tested to failure in shear.

Two different types of tests are made: (*a*) Normal tests, in which the water in the voids of the sample is under no excess pressure; and (*b*) rapid tests, in which the water carries part or all of the superimposed load.

In the first type, the normal water content under any given pressure is reached by allowing sufficient time for the pressure to force out all excess water. Depending upon the permeability of the soil, this process may require from an hour to several days. Among other things, the condition of normal water content is reached when the gradual compression of the sample ceases. When this condition prevails, the sample is sheared by gradually increasing

the horizontal force, Q, keeping the vertical pressure, P, constant. As may be seen from Fig. 45, the horizontal force is increased by adding water load at the end of a counterbalanced beam. This test is usually carried out under three different vertical pressures—1, 2, and 3 kg. per sq. cm.

The second type of test is started in the same way as the first. When the condition of normal water content is reached, however, the shearing test is not made. Instead, the vertical pressure is suddenly increased, thereby producing a condition of stress in the water. No time is allowed for this condition to adjust itself; on the contrary, the shearing operation is performed immediately, and as rapidly as possible.

This type of test is of considerable importance, since in many earth structures, such as canals, fluctuations of loading often occur so rapidly that the stresses produced in the water within the voids of the soil have no time to adjust themselves. The results of rapid shear tests, therefore, serve as a basis for analyzing the most unfavorable loading conditions.

Comparisons between the friction angles determined from the two types of test offer interesting indications concerning the character of the soil. To provide a numerical basis for such comparisons, a coefficient, λ_2, is defined by the relation,

$$\lambda_2 = \frac{\tan \mu'_{1+1}}{\tan \mu_2}$$

in which, μ_2 is the angle of internal friction determined from a normal test under a pressure of 2 kg. per sq. cm., and μ_{1+1} is the angle determined from a rapid test in which a pressure of 1 kg. per sq. cm. was applied; time was allowed for the stress in the water to adjust itself; then an additional load of 1 kg. per sq. cm. was applied, and the sample was sheared off immediately afterward.

The finer the grains of a soil, the smaller are the angles of internal friction, μ_{1+1} and μ_2, and the smaller is the coefficient, λ_2. For pure clays, λ_2 may attain very low values—perhaps in the neighborhood of 0.3—whereas for pure sands, or soils containing a large percentage of sand, it approaches unity.

Group (5).—The compressibility of the soil is measured by means of the apparatus shown in Fig. 46. The device is arranged so that the lateral pressure may be determined at the same time.

After the water content of the soil has been measured independently, the material is filled into the brass cylinder in a plastic condition, and the total weight is determined. Then a pressure of 1 kg. per sq. cm. is applied to the soil through a precisely machined piston. The compression, magnified ten times, is measured by the movement of the end of the load beam. Motion ceases when the water content is reduced to the normal value corresponding to the applied pressure. When this condition is reached a measurement of the lateral pressure is made. The horizontal force, H, is gradually diminished by siphoning water from a tank (not shown) until the index, Z, shows a small movement. The index is arranged to magnify the motion fifty times, because only a very small amount of movement is permissible if arching action and other disturbing effects are to be avoided.

Let σ_I and σ_{II} denote the vertical and horizontal pressures, respectively. Let β be their ratio, defined by $\beta = \frac{\sigma}{\sigma_I}$. From Mohr's diagram it is possible to determine the angle of internal friction, ϕ, as a function of β. The relation is $\tan \phi = \frac{1 - \beta}{2 \sqrt{\beta}}$.

After several measurements of the lateral pressure have been made, the weight of the cylinder and sample is again determined. The difference between this weight and the original weight represents the quantity of water

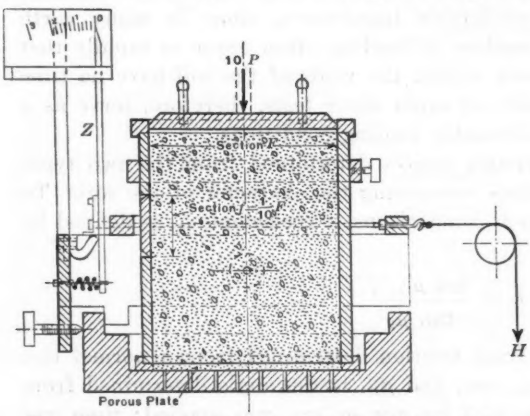

FIG. 46.—APPARATUS FOR CONSOLIDATION AND LATERAL PRESSURE TESTS.

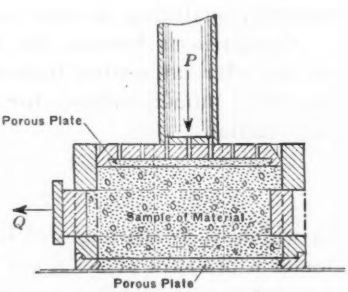

FIG. 47.—PROPOSED APPARATUS FOR CONSOLIDATION AND SHEARING TESTS.

forced out by the applied pressure. Next, the load is suddenly increased from 1 kg. per sq. cm. to 2 kg. per sq. cm. A lateral pressure measurement is made immediately, that is, while the water in the voids is still in a state of stress. Then follows a period of gradual compression under the increased load. When the compression is complete, the lateral pressure is determined once more. This continuous process is especially advantageous in that a whole series of measurements is made on the same sample without disturbing it in any way. Thus, a clear understanding of the relations between the various properties of the material is readily obtained.

The shearing apparatus, Fig. 45, and the consolidation apparatus, Fig. 46, can both be improved. The more favorable characteristics of both might be combined advantageously in a single device, such as that illustrated in Fig. 47.

Summary.—The foregoing description illustrates the possibility of making continuous tests, using relatively cheap and simple instruments right at the job, to measure the most important characteristics of the soils encountered in construction work. Without claiming that the results are highly accurate, the writer believes that field laboratories, designed to fulfill the needs of a single construction project, serve as a practical supplement to the investigations undertaken in scientific institutions. Such investigations should make it possible to improve upon the economy and safety of earth structures.

To illustrate the practical application of such studies, experience in connection with the power plant at Albbruck-Dogern, on the Rhine, and the Kockovce Lodce project, on the Waag, may be cited. In both cases the canals were lined with concrete to insure water-tightness (see Fig. 48). In both cases it was necessary to economize on gravel. Hence, the dikes had to be composed partly of silt. The question as to the best use of the available material was decided from the results of tests on the soil, and from statical computations based on the methods of Ehrenberg.[116]

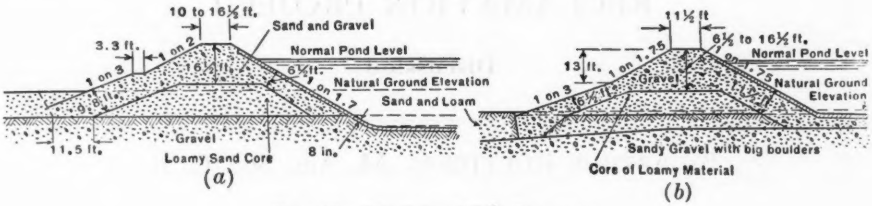

Fig. 48.—Typical Dike Sections.

Although the same factor of safety was applied to the slopes of both canals, the resulting cross-sections differed materially, as a comparison of Fig. 48(*a*) and Fig. 48(*b*) will show. In the case of the Waag project, the silt core had to be much smaller than that in the Albbruck-Dogern dikes, because the silt found in the Waag had a much smaller shearing resistance than the Rhine material.

The writer was assisted in all the tests by Mr. Robert Haefeli, who is in charge of the Albbruck-Dogern Laboratory and of similar laboratories on other projects with which the writer is associated.

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116 "Grundlagen der Berechnung von Staudämmen," *Wasserkraft und Wasserwirtschaft*, 1929, Heft 23.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

CONSTRUCTION WORK ON A FEDERAL RECLAMATION PROJECT

Discussion

BY ARTHUR RUETTGERS, M. Am. Soc. C. E.

ARTHUR RUETTGERS,[11] M. AM. SOC. C. E. (by letter).[11a]—This interesting paper is a commendable presentation of the salient construction features of the Kittitas Division of the Yakima Project—a work which included one of the most difficult and extraordinary pieces of canal construction ever undertaken by the U. S. Bureau of Reclamation.

Inasmuch as approximately one-half of the $9 000 000 investment was expended for concrete, the author fittingly emphasizes the importance of the procedure for controlling the quality of this material. The use of diatomaceous earth as a concrete admixture is briefly mentioned in the paper and the writer is taking this opportunity to submit some data on the subject.

The various concrete mixes established by laboratory tests and subsequently modified to some extent by early field experience on the Kittitas work have been presented previously in a paper by the writer and Mr. A. A. Whitmore.[12] No admixture was used in rich mixes having $4\frac{1}{2}$ or less parts of aggregate; $1\frac{1}{2}\%$ admixture was used in the 1:2:3.25 mix; and 3% admixture was used in mixes containing 6 parts or more of aggregate.

At first, diatomaceous silica was considered as a means of overcoming a noticeable harshness or lack of plasticity in the leaner laboratory mixes, including those that were over-sanded. This condition was attributed to a natural deficiency of fines in the sand and was alleviated by the admixture treatment, in so far as could be judged by the appearance, "feel," and finishing of the concrete. Before permitting the use of admixture in the field, however, the laboratory tests on plain concrete were extended to determine the effect of admixture on compressive strength for some of the more prac-

NOTE.—The paper by Morris Mason, Jun. Am. Soc. C. E., was published in October, 1931, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: December, 1931, by John Sanford Peck, Assoc. M. Am. Soc. C. E.; January, 1932, by Messrs. Edward W. Bush and Orrin H. Pilkey; and April, 1932, by Messrs. Clifford A. Betts, and George C. Imrie.

[11] With U. S. Bureau of Reclamation, Denver, Colo.

[11a] Received by the Secretary March 8, 1932.

[12] *Proceedings*, Am. Concrete Inst., Vol. 27, 1931, Table 1, p. 141.

ticable mixes. Comparisons were made on the basis of equivalent slumps, which resulted in generally higher water-cement ratios for the concrete with admixture. The average results of the 28-day strength tests are given in

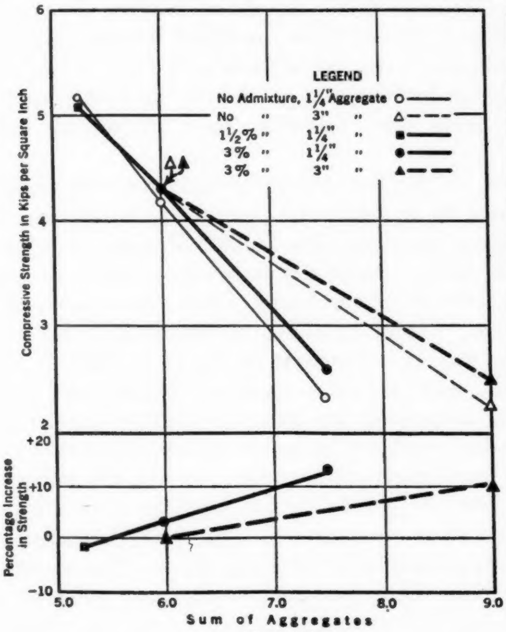

FIG. 23.

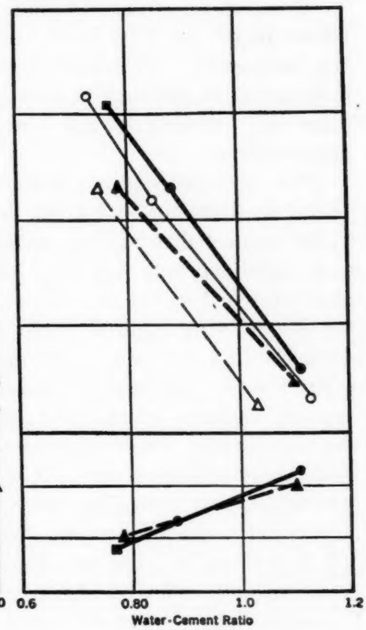

FIG. 24.

Table 6 and Figs. 23 and 24 and show that, with one exception, the strengths were increased from 1% to 15% by the use of admixture. It is interesting

TABLE 6.—STRENGTH TESTS

Sum of aggregates	Gravel to sand ratio	Slump, in inches	NO ADMIXTURE		WITH ADMIXTURE			
			Water-cement ratio	28-day strength in compression	Percentage, admixture	Water-cement ratio	28-day strength in compression	Percentage, increase in strength
FOR CONCRETE OF 1¼-INCH MAXIMUM SIZE OF AGGREGATE								
5.25.....	1.84	2	0.70	5 410	1.5	0.74	5 230	—3.3
5.25.....	1.84	6	0.76	4 910	1.5	0.80	4 955	0.9
6.0......	1.62	2	0.83	4 425	3	0.85	4 550	2.8
6.0......	1.62	6	0.87	3 935	3	0.91	4 045	2.8
7.5......	1.35	2	1.09	2 395	3	1.09	2 750	14.8
7.5......	1.35	6	1.17	2 330	3	1.16	2 570	10.3
FOR CONCRETE OF 3-INCH MAXIMUM SIZE OF AGGREGATE								
6.0......	2.28	2	0.75	4 290	3	0.78	4 305	0 4
9.0......	2.28	2	1.03	2 265	3	1.10	2 495	10.2

to note, in this connection, that the greater proportionate increases in strength. In these figures the sum of aggregates is by volume, dry rodded, in need of admixture to counteract harshness and tendency to segregation

were, fortunately, also the ones most greatly benefited from the standpoint of strength. In these figures the sum of aggregates is by volume, dry rodded, and the water-cement ratios are by volume.

Permeability tests, conducted principally for the determination of appropriate mixes for high-head concrete siphons, also furnished information on the comparative advantages of concrete with and without admixture. A brief description of these tests may be of interest, due to the fact that the procedure was materially different from that generally used in testing for permeability.

The test procedure consisted in forcing water through concrete disks 12 in. in diameter and 4 in. thick by applying air pressure to a column of water connected to the top surface of the disk. The volume of water entering each specimen was measured by noting water-surface readings on a gauge glass attached to the pipe water column. An ordinary water gauge was used for determining applied pressures. Except at night, on Sundays, and holidays, the flow into the specimens was maintained as nearly as practicable at a fixed rate, sufficient to exceed, slightly, the evaporation losses from the exposed surfaces of the disk, by increasing the pressure slightly whenever the periodically observed gauge-glass readings indicated a rate of inflow less than the established rate. The resulting pressure curve, except for porous specimens, and excluding the effect of periods outside working hours, was a gradually ascending one from zero pressure to (in some instances) 500 lb. per sq. in., the safe limiting capacity of the apparatus. Water outflow was collected and the accretions were measured at frequent intervals. Concrete disks were moist-cured for 14 days and then heat-dried to constant weight before being subjected to test.

A typical graphic test record, with gauge pressures, cumulative flows, and rates of flow plotted against time, is shown in Fig. 25. A photograph of the test apparatus is included in the paper previously cited.[13] The following test data apply to the typical case illustrated in Fig. 25: Real mix, 1:3.84; sum of aggregates (by volume), 4.50; nominal mix, 1:2.25:2.25; ratio of gravel to sand, 1.0; admixture in percentage (by weight) of cement, 0; slump, 6 in.; water-cement ratio (by volume), 0.70; average compressive strength (28 days), 4 645 lb. per sq. in.; maximum size of gravel, $1\frac{1}{4}$ in.; weight of disk at 14 days (moist), 39.56 lb.; weight of disk at test (heat-dried), 38.58 lb.; loss of water (by weight), 0.98 lb.; and loss of water, 444 cu. cm. The points plotted at breaks in the rate curves (Fig. 25(*a*)) represent average rates for the intervals preceding. Intervening points on the sloping connecting lines do not indicate actual rates. The line indicating the water-absorbing capacity of the disk is approximate, and is based on the loss in weight of the disk after it has been removed from the curing vat and heated. At Point 3, the total absorption was 1.22 lb., or 553 cu. cm., based on the increase in the weight of the disk at the end of the test.

Approximately fifty concrete disks, ranging from very rich to very lean mixes, were tested for permeability and in only two specimens was the avail-

[13] "Construction of Main Canal Lining on Kittitas Division, Yakima Reclamation Project, Washington," *Proceedings*, Am. Concrete Inst., Vol. 27, 1931, Fig. 20, p. 144.

able pressure of 500 lb. per sq. in. insufficient to produce outflow eventually. The average results of the tests, using "pressure to produce outflow" as a permeability index, are summarized in Table 7. As will be noted from the tabular values the resistance to permeation was markedly increased by the use

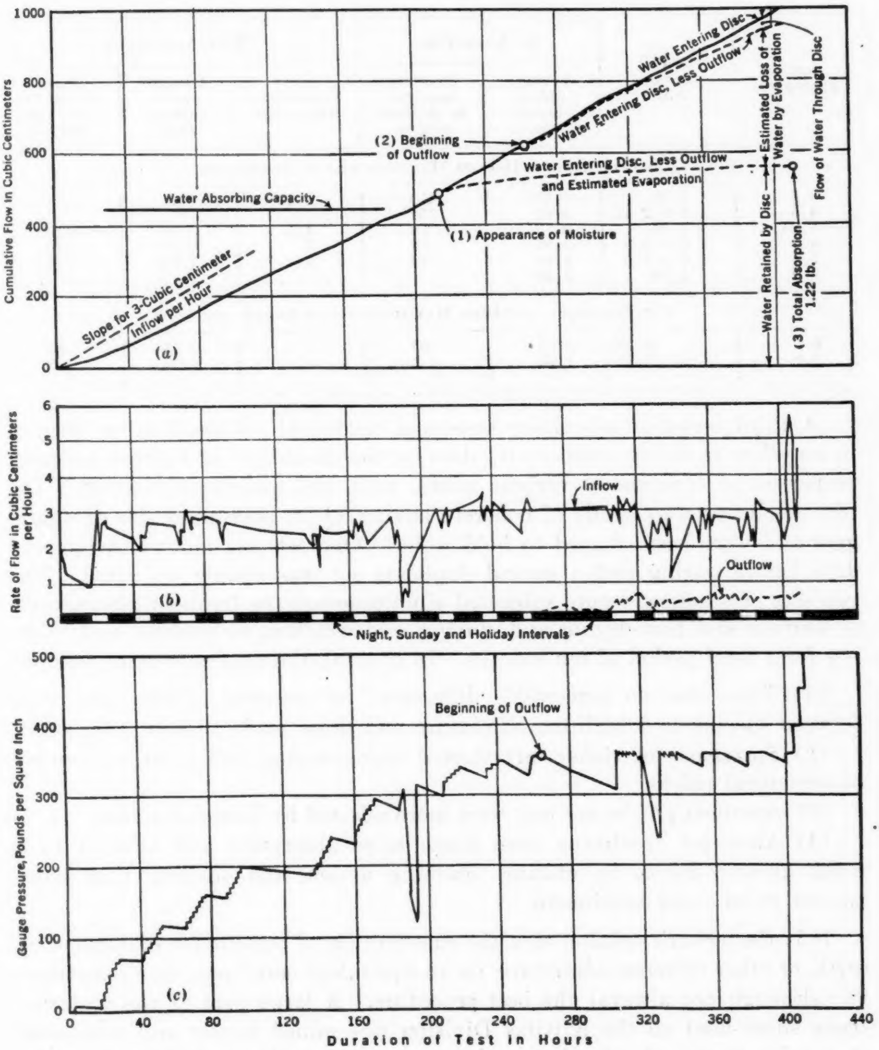

FIG. 25.—TYPICAL GRAPHICAL RECORD OF A PERMEABILITY TEST.

of admixture in concrete having $1\frac{1}{4}$-in. maximum size of aggregate, especially in the mixes containing a total of 6 parts of aggregate to 1 part of cement. The improvement was inappreciable, however, in concrete made with 3-in. maximum size of aggregate. These results would indicate that admixture is

most effective in promoting water-tightness where the voids in the concrete are relatively small in size and where large pieces of aggregate are not present to form direct water channels along their under surfaces.

TABLE 7.—PERMEABILITY TESTS

Sum of aggregates	Gravel to sand ratio	No Admixture		With Admixture		
		Average water-cement ratio	Pressure required to produce outflow	Percentage admixture	Average water-cement ratio	Pressure required to produce outflow
For Concrete of 1¼-Inch Maximum Size of Aggregate						
3.5......	2.83	0.575	410	....	....	....
4.5......	1.0 to 2.12	0.69	254	....	....	....
5.25.....	1.84	0.73	78	1.5	0.78	110
6.0......	1.0 to 2.0	0.87	75	3	0.92	310
7.5......	1.0 to 2.5	1.06	13	3	1.13	44
9.0......	1.18	1.39	1	....	....	....
For Concrete of 3-Inch Maximum Size of Aggregate						
6.0......	2.28	0.75	40	3	0.78	45
9.0......	2.28	1.03	2	3	1.06	6

A third series of laboratory tests was conducted on the Kittitas Project in an effort to obtain comparative data on the durability or weather-resisting properties of concrete of various mixes, with and without admixture. The test specimens were made of concrete having $1\frac{1}{4}$-in. maximum size of aggregate and were basin-shaped to hold water. One set was moist-cured for 14 days before testing and a second duplicate set was simply air-cured. The two sets of specimens were subjected simultaneously to frequent alternations of wetting and heat-drying and, during cold weather, to freezing and thawing for a total period of ten months. In general, the results were as follows:

(1) There was no perceptible difference, for concrete of the same mix, between specimens containing admixture and those made without admixture.

(2) Specimens of richer mix showed more crazing and a larger number of structural cracks.

(3) Specimens of leaner mix were more affected by leaching action.

(4) Air-cured specimens were much more absorptive and affected to a much greater degree by crazing, leaching action, and spalling than companion moist-cured specimens.

It is the writer's opinion that the substitution of cement for diatomaceous earth, or other siliceous admixture, on an equivalent cost basis, may sometimes be (although not always) the best procedure. A large part of the diatomaceous silica used on the Kittitas Division was mined locally and purchased at a price that made the delivered cost approximately 1 cent per lb. The delivered cost of cement was approximately 0.67 cent per lb., which means that $1\frac{1}{2}$ lb. of cement might have been used in lieu of each pound of diatomaceous earth admixture.

Considering the increased strengths obtained from the use of the diatomaceous silica, it seems reasonable to expect that the substitution of cement

would have yielded similar strength increases. On the other hand, the permeability test data available indicate that the improvement in water-tightness to be expected from the increased use of cement would have fallen far short of that accomplished by the introduction of diatomaceous silica. This is especially true, as previously pointed out, for the 1:6 mix which was used on most of the concrete work on the Kittitas Division. The substitution of cement would not have imparted those qualities of smoothness and uniform jelly-like consistency so characteristic of admixture-treated concrete and so helpful to a harsh mix in obtaining proper field placement and finishing. In this connection the writer is reminded of a preference expressed by one of the siphon contractors for the 1:2.0:3.25 mix with $1\frac{1}{2}\%$ admixture over the 1:1.7:2.8 mix without admixture, notwithstanding the fact that the latter mix is richer in cement by 14% and that both mixes have essentially the same gravel-to-sand ratio.

In conclusion, the writer does not wish to go one record as advocating the promiscuous use of diatomaceous earth or any other of the innumerable preparations of concrete admixture being promoted. However, some brands or types of admixture have a rightful place in concrete work and the problem is primarily one of determining experimentally their field of application and then applying such knowledge to the advantageous use of the materials in meeting individual requirements.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WESTERN HIGHWAY PRACTICE, WITH SPECIAL REFERENCE TO CALIFORNIA

Discussion

BY MESSRS. CHARLES T. LEEDS, AND G. S. PAXSON

CHARLES T. LEEDS,[8] M. AM. SOC. C. E. (by letter).[8a]—The problems of Western highway construction, particularly in the State of California, and the Southwest, are in many ways so different from those in other parts of the United States that this paper is extremely interesting. The California Division of Highways has much excellent work to its credit in the handling of difficulties such as those described.

Stream Erosion.—The problems of stream erosion, particularly those due to floods and cloudbursts, differ materially in California from those in the central and eastern sections of the country, because of the difference in climate. Many streams may be merely dry beds of sand in summer; and in winter they may become raging torrents. In desert sections of the country, as the author has stated, the water often appears without previous intimation that a cloudburst has occurred. The first knowledge that the observer has of approaching danger is the appearance of a wall of water and mud sweeping down the canyon. The writer cannot agree, however, with "the opinion of many engineers that these [cloudbursts] occur in approximately the same areas within a reasonable range of time." The writer has seen canyons from which no serious flood run-off is known to have occurred within historic times, and yet from which, within the last few years, sudden cloudbursts have brought down a literal wall of water, mud, and gravel, filling up the old channels and creating new ones, and spreading great fans of débris over the highway and the country below.

A serious difficulty of engineers in this Western country has been the paucity of records of rainfall and run-off upon which to base a reasonable

NOTE.—The paper by C. S. Pope, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, Sacramento, Calif., April 24, 1930, and published in November, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1932, by Messrs. J. S. Bright and R. D. Rader; and March, 1932, by Messrs. H. S. Kerr, E. Q. Sullivan, and T. A. Corry.

[8] Maj., Corps of Engrs., U. S. A. (*Retired*); Cons. Engr. (Quinton, Code and Hill-Leeds and Barnard), Los Angeles, Calif.

[8a] Received by the Secretary February 8, 1932.

estimate of probable flood flow for which provision should be made. In many cases it has been necessary to do as the railroad engineers were forced to do in early years when no records were available; namely, to make a personal inspection of the ground, determine the tributary area, guess at the probable maximum rainfall, and from that deduce a possible value for run-off. If the next flood proved the culvert or bridge inadequate, a larger one was built. This was not slipshod engineering, because no better fundamental data were available. It was the best possible under the circumstances. Now, however, the State Engineer's office maintains careful legal supervision of design and construction of all dams in California, and a painstaking study is being made of the rainfall and probable run-off in all the principal streams. Such data are of great value wherever it is necessary to erect a structure across or over any stream.

It is probably not out of place here to caution engineers against taking data from one stream or locality and carelessly applying them to another stream where actual run-off records are lacking. At first, the topographic conditions may seem similar in the two cases. Careful study, however, must be given to all available data on topography, wind direction, storm movement, elevation, and temperature, and their influence on precipitation.

Careful study of the geology of a section will often give valuable indication of the magnitude of past floods, and the débris of sand, cobbles, and boulders brought down thereby. The magnitude of a flood as it debouches from a canyon mouth may be very different from the magnitude of the same flood as it passes farther over the débris cone, or as it later reaches more level ground. In the first case, it is not merely a body of flowing water, but a moving mass of combined water, gravel, and sand, the larger and heavier portions of which will be deposited progressively as the flood proceeds downward, until, finally, there remains only the flowing water carrying fine sediment or silt.

Highway Location.—The problem of highway location through a desert region is not always a simple one. The closer to the mountains the location, the easier it will be to determine where the floods shall be carried across the highway, but the greater the mass of débris that will be brought thereto. On the other hand, if the highway is located farther away from the canyon mouths, the bridges or culverts may possibly be smaller, but the length of diversion dikes necessary to concentrate the broad sheets of water at control points may be greater.

An example of the difficulties experienced from wind and sand is that of the California State Highway passing through the Coachella Valley. The old county highway followed along the foot of the mountains on the southern border of the desert. This road was, of necessity, winding and crossed all the canyon mouths, but, at the same time, it was well sheltered from the severe windstorms which are not uncommon in that region. The new State Highway location, which passes down through the center of the valley, permits a much straighter road on a more even grade and requires fewer stream crossings. A saving of approximately five miles in distance and the elimination of practically all dips was effected thereby. Although the number of

bridges was unchanged, the general drainage conditions were greatly improved. On the other hand, in time of severe wind and consequent sandstorms, it is sometimes necessary to divert traffic from the main highway and cause it to pass along the old county road at the foot of the mountain where it is well sheltered. This is not so much because of drifting sand as because of wind velocity and sand-blast effect on automobiles.

Earthquakes.—The writer had the good (or bad) fortune to be in Santa Barbara at the time of the earthquake mentioned by the author, and can corroborate his statements that the concrete pavement was shaken from side to side until there was a gap of from 4 to 6 in. wide along each side of the highway. It had also buckled at a number of places and had settled at the bridge abutments. This action took place where the highway had been constructed on filled soft ground. He does not recall seeing any such evidence where the highway had been constructed on solid original ground. The very fact that the highway settled away from the bridge abutments to a distance of from 4 to 8 in. would seem to demonstrate that such settlement does not occur except where there is soft material underneath, and that, therefore, the highway fills should be constructed with the greatest solidity that is economically feasible. Even then some settlement is practically certain to occur.

The size and number of expansion joints in the pavement to prevent damage by earthquake must depend very largely upon the direction in which the probable earthquake wave will be propagated. If it is propagated longitudinally with the highway, expansion joints might take up this motion. On the other hand, if the direction of vibration is at right angles to the highway, it is not seen how expansion joints would be of any advantage. This, again, would seem to be evident from the observations in the Santa Barbara earthquake. Expansion joints would not have taken care of the kind of damage cited by the author, and no serious harm was done. If this earthquake wave (which apparently may have had an amplitude of vibration of from 8 to 12 in. at right angles to the highway axis) had progressed longitudinally of the highway, it is interesting to inquire what width of expansion joints should be inserted in the pavement and at what intervals to resist such action.

The entire question is well summed up by the author in the statement that "generally speaking, however, the danger due to earthquakes is not serious so far as highways are concerned." In view of the probable long intervals between earthquakes in California, it is not believed advisable or economically justifiable, to take undue measures to protect against their occurrence on highway construction.

Foundations.—One phase of the subject upon which the author has not touched is that of foundations for highway structures crossing streams. This phase is particularly important in a country such as California, subject to heavy floods, where stream beds may be composed of sand or silt to a very considerable depth. In the case of pile bridges it is not always adequate simply to drive the piling to a depth sufficient to secure adequate carrying capacity because, in time of flood, the bed of sand in the river may become

so saturated with water that the entire stream bed is moving down stream. This has been observed in floods in both the Los Angeles and San Gabriel Rivers. In the flood of 1889-90, the pile trestle bridge that carried the Southern Pacific Railroad across the San Gabriel River moved bodily down stream several feet without breaking or turning over, the entire railroad being simply carried down stream laterally that distance.

It is very common in some streams, as in the Colorado River, for the bed to scour to extreme depth in time of flood and then to refill as the flood peak passes, so that the actual depth of the stream at the time of maximum flood is far greater than any measurements after the flood may indicate. The failure of the highway toll bridge across the Colorado River, from Blythe, Calif., to Ehrenberg, Ariz., in 1928, was largely due to inadequate precaution against scour around the bridge piers, which scour occurred to much greater depth than had been anticipated.

River Crossings.—Because many streams in California, particularly in their higher sections, are to some extent "riding on a ridge," it is very necessary, in designing a river crossing, that study should be made not only of the immediate locality, but of the entire river higher up, in order that adequate measures may be taken to assure that the river shall always pass under the bridge to be erected. In 1906, two bridges were constructed across parallel channels of the San Gabriel River on its débris cone, not far from the mouth of the canyon. They had ample flood capacity, but the flood of 1914 instead of passing under them as had been anticipated, cut new channels for itself, still farther apart, tearing out the outer approaches and thus leaving the bridges "high and dry." Since then these bridges have been replaced by a modern concrete bridge and adequate precautions have been taken to protect the banks thoroughly above the bridge, and a repetition of such a disaster has been prevented.

Bank Protection.—The author has referred to the flood in the Santa Clara River, caused by the breaking of the St. Francis Dam. This was, of course, comparable to a cloudburst and of far greater dimensions. As a result, much damage was done in the Santa Clara Valley and considerable bank protection previously constructed was either damaged or destroyed. As an aftermath, the City of Los Angeles, which had constructed this dam, not only paid for all damages which had resulted, but also appointed a consulting board to work with a similar engineering board, representing Ventura County, in preparing plans for bank protection which should afford to all owners of abutting property the same measure of protection which they had prior to the flood.

In the case of pre-existing protection works, the aim was to repair them to their former efficiency where the protection was not wholly destroyed and, where it was entirely destroyed, to construct new works of similar character to those which existed before the flood, or works of an equivalent efficiency. In the case of new work the problem resolved itself into: (1) The construction of levees to keep the river within reasonable limits of width, the levees to be protected by a system of spurs of different types of construction, depending on the conditions encountered; and (2) where no levees were

required, the construction of spurs to protect the existing banks. Wherever levees were constructed, it was required that, in order to prevent the formation of a channel along the toe of the levee, the borrow-pit should not be continuous along the levee. A bar was left between the protecting spurs not less than 20 ft. wide. Where no spurs were required, the bar was to be left at intervals no greater than 200 ft. apart.

Three types of protection for levees or banks were proposed, designated as Types *A*, *B*, and *C*.

Type *A* (see Fig. 18) composed of creosoted timber piles, was used as a replacement for similar protection which had been destroyed. It consisted

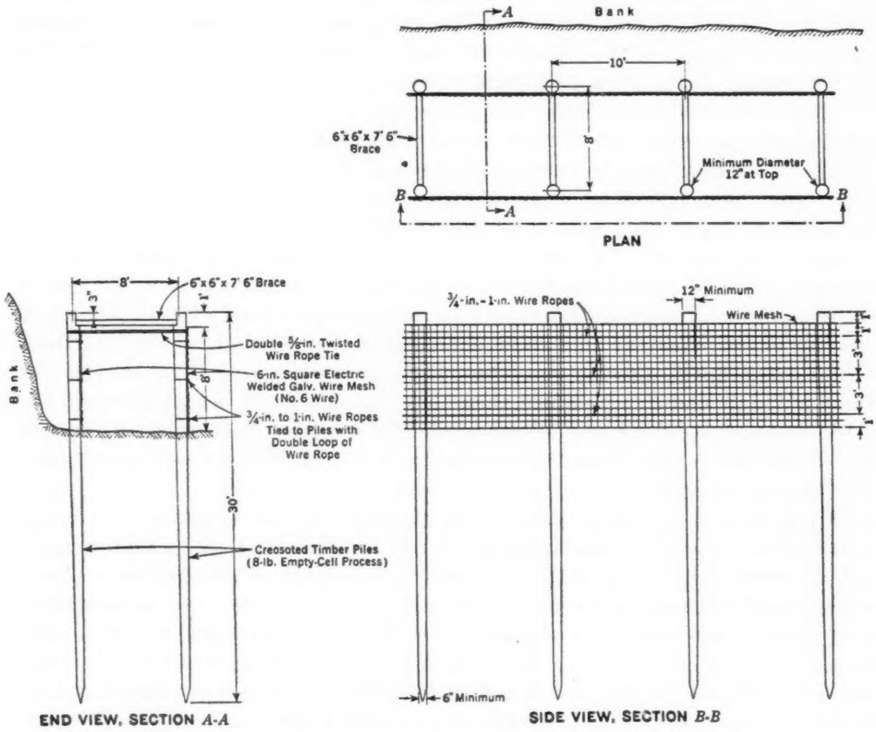

FIG. 18.—TYPE *A* BANK PROTECTION.

of timber piles driven in two rows 8 ft. apart, 10 ft., center to center, in rows, and braced between with a 6 by 6 in. by 7 ft. 6-in. timber near the top.

These piles were not less than 12 in. in diameter at the butt, 6 in. in diameter at the tip, 30 ft. long, and were driven to a penetration of not less than 16 ft. and where possible to such penetration that their tops extended 9 ft. above scour grade of the river. This dimension was governed, of course, by the cross-section of the river. The piles were of Douglas fir and creosoted by the 8-lb. empty-cell process. Three 1-in, wire ropes were used in each row to tie the piles together. These ropes were stretched taut against the piles

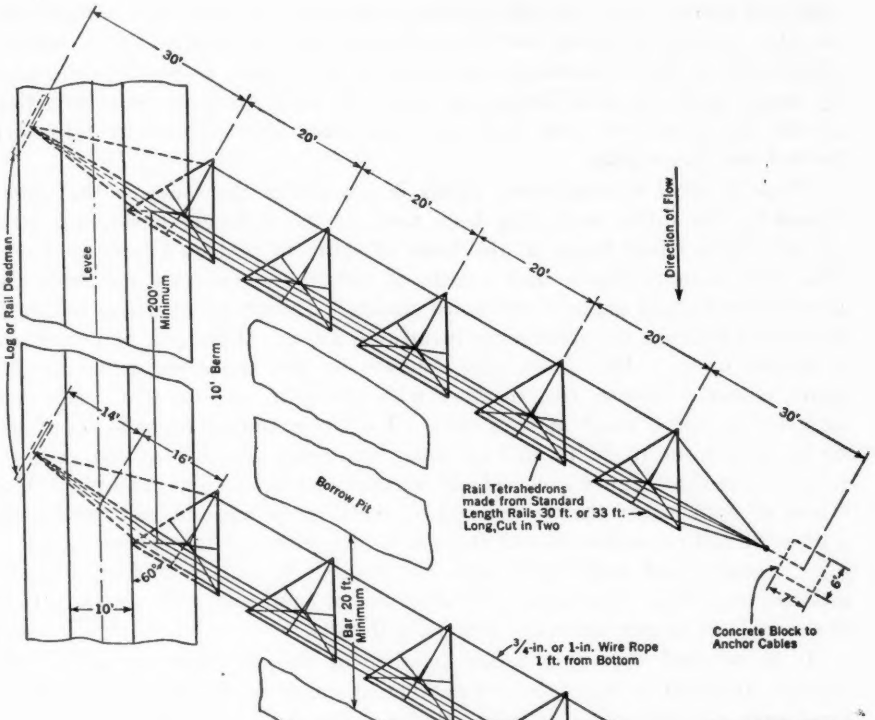

FIG. 19.—BANK PROTECTION TYPE *B*.

and tied thereto with an independent short piece of wire rope. Each row of piles was also faced on the river side with an 8-ft. width of 6-in. square, electrically-welded, galvanized wire mesh of No. 6 wire, fastened to the piles by heavy staples at each horizontal wire. To hold the piles from spreading at the top, a loop of ⅝-in. wire rope was taken around opposite piles and twisted together tightly.

Type *B* (rail tetrahedrons) which is essentially the same as that mentioned by the author as having been used on the Colorado River, was used on the Santa Clara River in the form of spurs to protect a levee or bank. They were constructed in unit lengths of 100 ft. and extended out from the levee or bank at an angle of 60° down stream. Except as otherwise specified, the spaces between the spurs were invariably 200 ft. This type of protection is shown in Fig. 19. Each unit consists of five tetrahedrons set 20 ft. apart, center to center, tied together with five ¾-in. to 1-in. wire ropes and anchored to "dead men" at both ends. Each tetrahedron is made up of six 60-lb. rails from 15 ft. to 16.5 ft. long, connected together at the ends to form a four-sided figure each side of which is an equilateral triangle. The center of each leg of the triangle which rests on the ground, is braced with a piece of rail approximately 12 ft. long to the center of the opposite sloping leg. Second-hand rails were used for this work and were found amply satisfactory. The wire ropes were also second-hand and were securely tied to the rails at proper intervals with ¾-in. U-bolts.

It is believed that this tying together of the tetrahedrons with four cables, 1 ft. apart, on the face and another at the back, is a distinct improvement over the protection as used on the Colorado River. Each spur unit was anchored at each end. The anchor at the levee end of the spur consisted of a sound log or two logs of a combined minimum cross-section of 1.5 sq. ft. and not less than 12 ft. long, buried 6 ft. in the ground. The ends of the wire rope tying the tetrahedrons together were looped around the anchor and fastened back on the rope with bull-dog wire-rope clips, two to each rope. The anchor at the river end of the spur consisted of a rubble concrete block, 5 cu. yd. in volume. The block was set 12 ft. in the ground and properly anchored in it was a 12-ft. length of 60-lb. rail, to the upper end of which the wire rope tying the tetrahedrons together was fastened.

This type of protection was used in lieu of Types *A* and *C* where the required pile penetration could not be secured.

Type *C*, a system of iron-pipe piling, was used in a manner similar to Type *B* to protect levees and banks and, as in Type *B*, was used in the form of spurs, with one exception. This exception, for protection work opposite the City of Santa Paula, was a location where the river bank and levee on the edge thereof, were protected for a distance of 3 000 ft. by a continuous row of Type *B* construction in addition to spurs.

As in Type *B*, the spurs were each 100 ft. long and extended from the protected levee or bank at an angle of 60° down stream and were spaced 200 ft. apart (see Fig. 20). Each spur is 100 ft. long and consists of a double row of pipe piling, driven 10 ft. on centers in rows 8 ft. apart. The piles

used in the lower half or river end of the spur are 6 in. in diameter, while those for the upper half, or levee end, are 4 in. in diameter.

The 6-in. pipe piling, constituting the river end of the spur, was driven to a minimum penetration of 16 ft., or to a penetration of 18 ft. if refusal was not reached before that depth. The tops of the piles were on a straight line sloping downward toward the river on a 2% grade. The 4-in. pipe

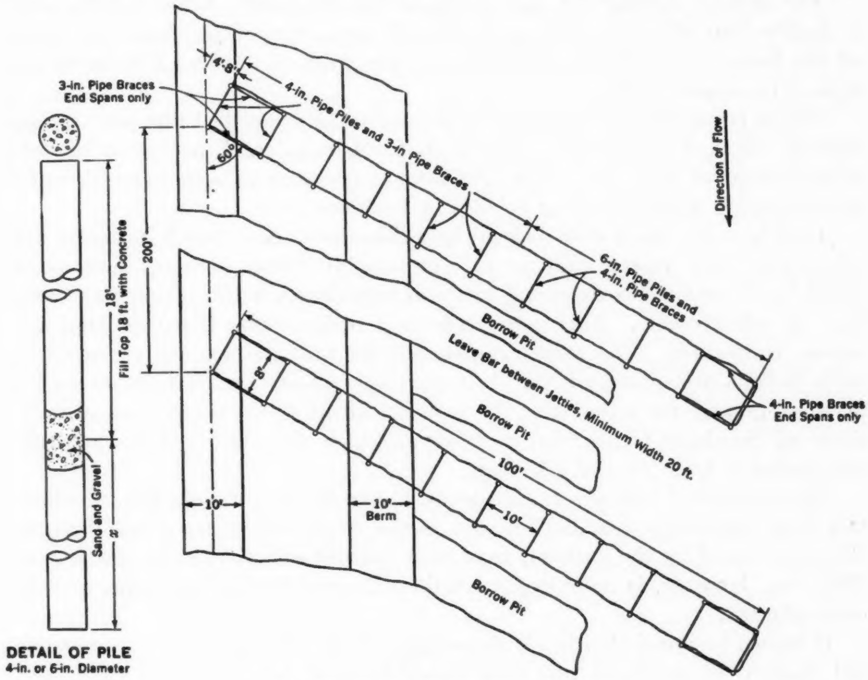

FIG. 20.—BANK PROTECTION, TYPE *C*.

piling, constituting the levee end of the spur, was driven to a minimum penetration of 14 ft. For economy, second-hand pipe was used throughout the job, subject to inspection and approval.

Between rows the piles were braced by welding a piece of pipe between opposite piles, 2 ft. from the top. The two piles on each end of the spur were also braced longitudinally to the second pile from the end by welding pipe horizontally between them, 2 ft. from the top.

For greater rigidity the last four piles on the end were tied together diagonally by $\frac{5}{8}$-in. wire rope twisted tightly between the second pile from the end, 2 ft. from the top, and the end piles, 8 ft. from the top. The rows of piling on the up-stream side of the spur were tied together longitudinally with two second-hand wire ropes of $\frac{3}{4}$ in. to 1 in. in diameter, well tied to the piles with No. 6, annealed, galvanized wire.

Each row of piling on the up-stream side was faced with an 8-ft. width of 6-in. square, electrically welded, wire mesh of No. 6 galvanized wire, stretched taut and securely tied to the piles with a double loop of No. 6

annealed galvanized wire at all horizontal wires. The top of the wire mesh was flush with the tops of the piles. The wire mesh on the up-stream row of piles was extended around the river end of the spur, and securely tied to the first pile of the back row. The wire mesh was also tied with No. 6 galvanized wire, at 12-in. intervals, to the wire ropes on the up-stream row of piling.

For greater rigidity the piling was braced diagonally between rows with a double line of $\frac{5}{8}$-in. wire rope, twisted tight, extending from the point of the brace weld 2 ft. from the tops of the piles to a point 8 ft. from the tops of the opposite piles.

Where it was found impracticable to secure the prescribed pile penetration, namely, 16 ft. for 6-in. piles and 14 ft. for 4-in. piles, Type *B* spurs were used instead of Type *C*. This was found necessary in some cases because of underlying layers of heavy cobbles or boulders.

In this work, brush and rock-fill spur-dikes were considered, but were not adopted for the reason that in this climate all brush becomes thoroughly dried in the summer season and hence it constitutes a fire menace and also loses its effectiveness. This is not true in a region where it can be kept wet almost constantly. The same argument holds true for any willow mattress work in bank protection, which, while extremely effective on rivers such as the Mississippi and its tributaries, is entirely unsuitable on streams, such as those of Southern California or Arizona, which are almost if not entirely dry during a large part of the year.

The reinforced concrete tetrahedrons, shown in the author's Fig. 6, which had been previously installed on the Santa Clara River (not the Ventura River, as stated by the author), have been entirely effective, as he has stated. This type, however, is more expensive than the rail tetrahedron type, and no more efficient.

It should be noted that in all these types of spur-dike protection, an essential feature of merit is provided by cables and wire netting which are stretched along the up-stream side. The function of a spur-dike, when used for bank protection, is to deflect currents and also to retard the velocity of the water and thereby produce a deposition of silt behind the spur. It is, therefore, highly essential that these spurs should be constructed so as to catch drift and thereby increase the resistance to flow of the river. Where the river does not carry any appreciable drift, therefore, such spurs will not be as effective as where considerable floating brush and leaves will lodge against the spurs and thereby increase their effectiveness. It is also highly essential as the author intimates under "Jetties," that the spur be constructed so as to prevent possibility of scouring under the fencing. Such action, if it occurs, may be more injurious than if no spurs had been constructed, for the undercut spur concentrates the scour.

Protection by Sea-Walls.—The writer had occasion to observe the protection work of reinforced concrete cells, so well described by the author, while they were under construction by the California Highway Division, and he desires to register his commendation of this excellent device in a difficult situation.

The sea-walls constructed by the State of California of monolithic concrete appear to be of sufficient strength to withstand the action of the heaviest storm waves thus far encountered. The writer believes, however, that the profile adopted has had a tendency, in some instances, to erode the beach, and that with a different design a considerably greater effective beach width could have been secured.

It is true, as the author states, that the use of sheet-piling, of either wood, steel, or concrete, on State work in California has never met with much success, and that it has not been used extensively in sea protection. Except in rare cases, it should not be so used. When a vertical wall is built to offer resistance to the onrush of the waves, the inevitable result is a downward deflection of part of the wave and a throwing into the air of a mass of water which, dropping again with great force, scours out the sand at the toe of the wall. The profile of the wall should be so designed, with slope and curve, as to deliver the return wash of the wave parallel to the beach slope, or still better, to provide a means of breaking up the force of the wave, as with rip-rap, or with a step-faced sea-wall of adequate design.

Shore Protection by Groynes.—Groynes for the protection or improvement of the beach line are extremely effective where properly used. Groynes of steel sheet-piling or of interlocking concrete blocks have usually been more effective than those of wood, however. Wood groynes, too, are subject to damage by storms. It is often difficult to secure sufficient penetration for the vertical wood piling to prevent their being worked loose in severe storms. Horizontal planking is often torn loose and is quite likely to have the beach eroded below the level of the lowest plank under severe storm conditions. If any opening occurs below the bottom plank, the force of the water flowing under it produces a severe jet action which scours out the beach to a much greater depth than if the groyne had not been in existence. With steel sheet-piling or concrete blocks that can settle into place, there is no danger of this.

In the location of groynes the simplest rule (and the one most generally followed) is to construct them at right angles to the shore line. This, however, is not the scientific procedure, and the groynes will be found to function more effectively if their orientation is carefully determined from a study of the prevalent direction of wave action on the shore. The height, length, and profile of the groyne are also important elements which can only be determined from a careful study of local conditions.

At the present time (1932) the California Division of Highways, and the California Division of State Lands are co-operating in planning a system of steel sheet-pile groyne protection for a section of the State Highway where there has been severe erosion. In this study, the inter-related effects of adjacent parts of the shore line and previous construction thereon have been carefully considered, and the future results will be noted.

This entire problem of shore protection and beach improvement is an intricate one in which no isolated problem can be adequately solved without consideration of adjoining section of the coast. It is probable that in the past adequate attention and study have not been given to this in many

sections of the country. Much can be done, however, by thoroughly co-ordinating all efforts of State, municipal, and private agencies. This is particularly important in connection with highway construction, as a large measure of the usefulness of the highways is as a means of access to and along the beach and ocean by people of the hinterland.

G. S. Paxson,[9] Assoc. M. Am. Soc. C. E. (by letter).[9a]—Except the works designed to withstand earthquake shocks, the problems discussed by Mr. Pope are common to all the Western States. Several interesting and successful structures, designed to protect highways from stream erosion and from the encroachment of sand dunes, have been built in Oregon in recent years.

The State has several rivers of medium size (approximately 10 000 sec-ft. maximum discharge) that flow through flat alluvial plains. The banks of these rivers are composed of sand and gravel and are eroded easily. The rivers are fed by smaller streams flowing on steep gradients and carrying large quantities of sediment. On joining the main river the rate of flow becomes less and the stream contains more sediment than can be carried at the lower velocity. Under these conditions bars are formed that often change the direction of the current causing serious erosion along the banks. To control the current before it reaches the stream bank is usually more economical than to reinforce the banks themselves. A cheap and efficient current-directing structure is shown in Fig. 21. It is built of rock enclosed in wire mesh, and is most easily constructed during the low-water period when the site is dry. A width of wire mesh is laid in position on the stream bed and rock is piled on it to form a dike the section of which is an equilateral triangle. Other sections of mesh are laced to the edges of the bottom section, folded over the sides of the dike, and laced together at the top. The dike is thus completely enclosed by mesh and will not break up even if it is undermined. Dikes of this type have rolled completely over without damage. The crest is built to about the ordinary winter water elevation. During flood periods the current is checked enough to cause sediment to become deposited behind the dike. In a few years an elevated plain is built up, that acts as a buffer between the stream and the threatened bank. This type of current retarder built 5 ft. high, costs about $4 per lin. ft.

This treatment has not been applied to streams that have a rise of more than 8 or 10 ft. during flood periods. For such streams the usual treatment has been a bank protection consisting of piling, brush fascines, and rock. Fig. 22 shows a revetment on the bank of the Willamette River near Harrisburg, Ore. The piles in the front row are spaced at 4 ft. 0-in. centers. The back row is 8 ft. 0 in. from the front row, and the piles are at 8 ft. 0-in. centers. The space between the rows of piles is filled with willows tied in bundles 8 ft. long and 2 ft. in diameter. These fascines are weighted with large blocks of stone. Where the revetment is in contact with the bank many of the willows take root and grow. In a short time a compact and resistant barrier is formed.

[9] Asst. Bridge Engr., State Highway Dept., Salem, Ore.

[9a] Received by the Secretary February 11, 1932.

Fig. 21.—Current-Directing Structure of Rock Enclosed in Wire Mesh.

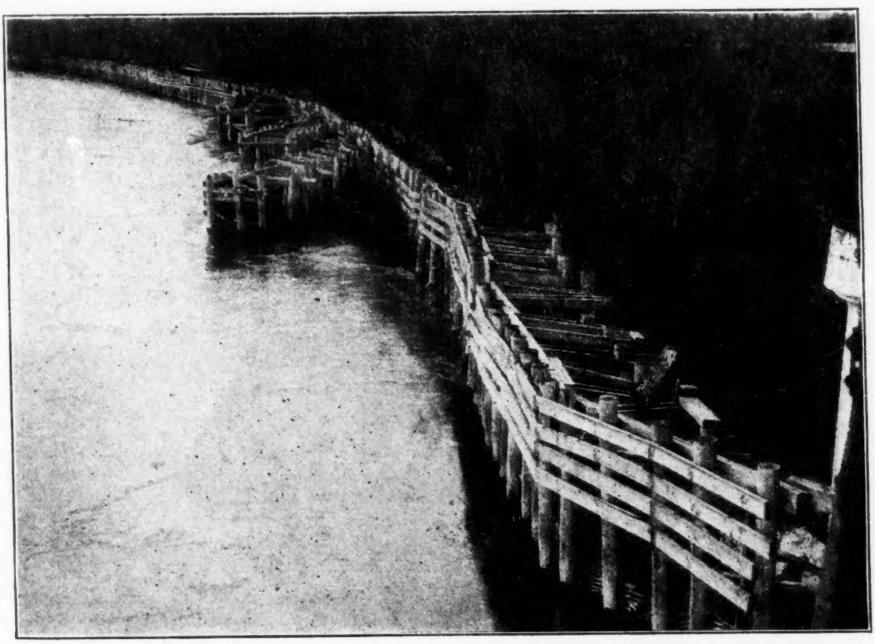

Fig. 22.—Bank Revetment of Piling, Brush Fascines, and Rock on Willamette River, Oregon.

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In many places, highways have been built through or beside sand dunes that shift on to the roadway during storms. The removal of this sand is a continuous and costly maintenance operation and many attempts have been made to control it. The most successful has been the planting of grass or shrubs. For example, "Holland grass," which grows wild along the Oregon Coast, is planted in bunches of three or four stalks at 2-ft. centers. It has an extensive root system which holds the sand in place and the tops act as a wind-break. This grass is quite hardy and survives the dry summer months with little damage. Successful plantings have also been made with Maritime pine, an importation from France, and with native shrubs, such as rhododendron, huckleberry, and salal.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

EFFECTS OF BENDING WIRE ROPE

Discussion

By Messrs. O. P. Erickson, F. W. Deck, and O. G. Julian and J. C. Damon

O. P. Erickson,[20] M. Am. Soc. C. E. (by letter).[20a]—This is a timely paper on a product of wide use in engineering construction and operation activities. That considerable difference of opinion exists regarding the stresses in wire rope is indicated by the many formulas encountered in wire rope literature. As shown by the author,[20b] by R. C. Strachan,[21] M. Am. Soc. C. E., and others, none of these formulas agrees even approximately with each other. The reason for this is the many variables involved, the most important being lubrication, size arrangement and condition of sheaves and drums, type and size of rope center, length of strand, strand construction, and rope lays. Mr. J. F. Howe[22] states that the modulus of elasticity for high-grade rope wire varies between 25 500 000 and 29 000 000. For stressed ropes, he gives modulus values ranging from 7 000 000 to 13 000 000, depending on the construction of the rope as an average of test results, and states that for new unstressed ropes these values will be somewhat reduced. The foregoing makes the selection of a piece of wire rope seem a formidable task.

Actually, however, rope users can obtain considerable practical information from manufacturers' catalogues. The wire rope user and his consulting engineer can also avail themselves of the rope maker's wide experience regarding sheave and drum diameters, rope leads, and type of rope to use. This information, which may be both theoretical and practical, is freely given, and the rope maker because he is consulted on the problem will make a special effort to furnish the best possible type of rope and service. On the other hand, if the rope user and his consultant specifies all the physical properties of the rope and its application, the rope maker may feel, and

Note.—The paper by Frederick C. Carstarphen, M. Am. Soc. C. E., was published in December, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1932, by Messrs. Robert C. Strachan, and C. D. Meals.

[20] Chicago, Ill.

[20a] Received by the Secretary February 26, 1932.

[20b] *Proceedings*, Am. Soc. C. E., December, 1931, p. 1456.

[21] *Civil Engineering*, November, 1930, p. 111.

[22] *Transactions*, Am. Soc. Mech. Engrs., 1918, p. 1061.

perhaps rightly so, that his responsibility ends on delivery of the rope. Therefore, in order to insure better service co-operation between the rope user and the rope maker, the specifications should be made sufficiently flexible to cover the operating life of the rope, and the rope should be paid for according to its hours of service.

As pointed out by rope makers, and confirmed by the experience of most rope users, the importance of proper lubrication cannot be over-emphasized. The proper working load of a well lubricated rope running over a sheave may become a dangerous over-load for the same rope when devoid of lubrication. Ropes subjected to temporary over-loads or under-water usage require much more than the average lubrication due to the lubricant being squeezed out or washed away. In order to penetrate between the wires to the rope core most lubricants must be applied hot. Unless they penetrate between the wires they are of little value in prolonging the rope life.

Other causes for exceedingly short rope life are too small sheaves and reverse bends. The U. S. Government Master Specifications for Wire Rope,[23] gives the following sheave diameters: For 6×19 rope construction good practice requires the sheave to be 50 rope diameters, and under no circumstances less than 20 rope diameters. For 6×37 or 8×19 rope construction good practice requires the sheave to be 30 rope diameters, and under no circumstances less than 15 rope diameters. If reverse bending cannot be avoided the sheave about which it occurs, if possible, should be larger than the other sheaves. This applies to bends of even small angularity.

F. W. Deck,[24] Assoc. M. Am. Soc. C. E. (by letter).[24a]—The author's method of attacking the problem of the effects of bending should serve to clear up many of the difficulties that attend the study of wire rope, and all kinds of stranded cables that act under bending similarly to wire rope. A complete understanding of what takes place when a stranded cable is bent is necessary not only in considering the more violent and acute bendings that take place, such as those in wire ropes used for hoisting, cableways, elevators, etc., but also in the study of the high frequency vibration that occurs in ropes and cables. This latter action causes a repetitive bending, usually of low amplitude and high frequency, which, if it occurs at a point of restriction to the free movement of the strands, may soon cause failure through fatigue in the metal of which the strands are composed.

Failures of this kind are experienced in many cases of cables used in the transmission of electric energy, or under similar conditions of usage, where they may be subjected to the constant action of wind or other forces tending to set up a condition of resonance. Under the condition of free vibration, the cable does not usually sustain serious injury, but where the cable is restrained, as at its terminal or clamp, the free slippage of strands and the stress equalization due to stranding are not allowed to take place, and, consequently, the bending becomes excessive.

[23] No. 297, May, 1925, p. 32.

[24] Structural Engr., Mech. Div., Philadelphia Elec. Co., Philadelphia, Pa.

[24a] Received by the Secretary March 3, 1932.

In any study of this action, an analysis of the effects of bending, such as is described by the author, is of material service. While the bending in vibration is slight as compared with bending around a sheave, the incipent action of the strands is similar. In the vibrational bending, it is consistent to use the beam elastic curve analysis; while in bending about a sheave, as the author points out, this principle may lead to serious error in calculations.

The experience which some public utilities have had in the use of wire rope bears out many of Mr. Carstarphen's statements. One of the primary uses to which wire rope is subjected in the public utility service is the handling of coal at steam-generating stations. This involves heavy loads, high accelerations, and severe abrasive action. Among the factors that are most important in contributing to rope failures, is friction. Abrasion nearly always precedes the breaking of wires, and practically none of the reduction in size is due to corrosion, which is almost unknown in coal-handling service. The bending, tensile, and shearing stresses are severely augmented by small sheaves, small drums, and reverse bends, together with rapid acceleration and deceleration of the loads and improper reeving of the rope.

Proper size and maintenance of sheaves have naturally been found to be one solution in increasing wire-rope life. Proper lubrication and care in handling are probably the most improtant factors, after care has been taken of the mechanical features of the equipment. The most important of these (care in handling) can be obtained only by careful attention to the operating personnel that handles the rope. It has been found possible to increase the life of rope several times by introducing a spirit of responsibility and competition among the men whose duty it is to use it. Consciousness on the part of the personnel that rope made by reliable manufacturers will give long life, and that early faults can not always be ascribed to poor manufacture, but are usually due to improper usage, will serve to increase such life to a rather remarkable degree.

Specifically commenting on the mathematics used by the author, the writer suggests revision of the substantiating statements in Example 1 as follows: If the two wires are stressed, and if no slipping occurs between them, the bent wire will be elongated and will rotate about A so that its end, C, will take the new position, B'; but if no slipping occurs, there is no increase in length, F (Fig. 12), which remains $2\pi d'$, and hence dl_1 becomes equal to the distance, AB', minus AC, while the angle decreases. The increment, dl_1, will always be less than dl', because it is one of the legs of an approximate right triangle, while dl' is the hypotenuse. Hence, the true ratio of stresses,

$$\frac{T_1}{T'} = \frac{\frac{dl_1}{l_1}}{\frac{dl'}{l'}} = \frac{l'\,dl_1}{l_1\,d\,l'}$$; therefore, T_1 is less than T', or the reverse of the condition specified by the author. It is not the inner wire that fails to take its full share of stress to prevent overstressing the outer wires; it is the outer wires, which are not stressed fully to avoid overstressing in the inner wire.

Furthermore with the symbols used by the author, if $\dfrac{dl_1}{dl'} = \dfrac{l_1}{l'}$ then $\dfrac{dl_1}{l_1} = \dfrac{dl'}{l'}$ and $T_1 = T'$ and, hence, $\dfrac{T_1}{T'}$ cannot equal 1.038. The results are similar, in giving a strand of lesser strength than the sum of the individual strengths of its wires.

The writer feels that there is some confusion of symbols in Equation (1) and the substantiating equation; and that here, as in other places, a more complete explanation, mathematical or descriptive, of the processes accomplished would aid greatly in clarifying the author's meaning. While all readers may

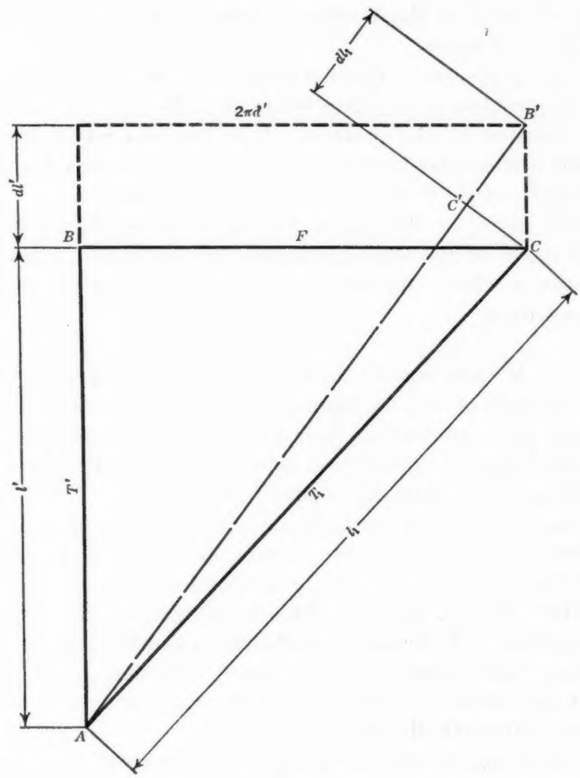

FIG. 12.

not desire to check, in detail, the formulas derived, yet when so much of the derivation of important formulas is presented, care should be taken not to omit those connecting links which give smooth understanding of the author's meaning.

The author deprecates the use in the present formulas of the expression for the elasticity curve in beams; and then, apparently, without sufficient explanation or reasons, he utilizes the same terms and the same basis in

Equations (13) to (17) for the potential energy of bending and twisting. These formulas can be used because of the application to a single wire which takes the characteristics of a bar with a constant modulus and a relatively small diameter. However, Mr. Carstarphen has neglected to make this fact clear, together with the impossibility of using the elastic curve formula for the cable as a whole on account of the flow of material and stress in the wires and strands during bending. This latter condition allows the bending of a stranded cable without excessive stresses, whereas a single bar of larger diameter could not approach such a condition of bending. Changes in the value of the assumed modulus of elasticity as made in some of the formulas in current use cannot compensate for this flow of material; nor can such formulas take account of the frictional restriction to motion and bending which causes the real stress.

The author has performed a distinct service to wire rope users in his method of attacking the problem of bending wire rope. He has eliminated from his mathematical solution of the problem, those features which would tend to lead the analyst astray, and has given a reasonable formula for ascertaining the loss in strength of the wire when bent around a sheave.

The formulas given in this paper for the bending of ordinary stranded cables and the study of the elasticity curve for locked-coil cables, should be of great assistance when combined with tabulated results of experiments along these same lines.

O. G. JULIAN,[25] M. AM. SOC. C. E., and J. C. DAMON[26] ESQ. (by letter).[26a]—An interesting account of the development and use of ropes and an analysis of certain problems are contained in this paper. The writers appreciate Mr. Carstarphen's initiative in taking up a subject on which little has been written, and regarding which exact knowledge is scarce.

Regarding the modulus of elasticity of steel the author states (in commenting on Table 1) that, "this factor varies, of course, depending upon the composition of the steel, but will usually be found to be between the limits of 25 000 and 30 000 kips per sq. in." This is contrary to the writers' understanding. The late George F. Swain, Past-President and Hon. M. Am. Soc. C. E., has stated[27] that, "notwithstanding the great variation in strength, it is remarkable that the modulus of elasticity of all grades of steel remains nearly constant at about 30 000 000 lb. per sq. in."

The section of the paper entitled 'Analysis of Strength" is interesting and the numerical value obtained for the ratio of the strength of a strand to the sum of the strengths of the individual wires making up the strand (which ratio may be termed the "efficiency" of the strand), agrees well with known values obtained from tests; however, the writers have some qualms regarding the method Mr. Carstarphen uses to obtain these values.

[25] Engr., Jackson & Moreland, Boston, Mass.

[26] Engr., Jackson & Moreland, Boston, Mass.

[26a] Received by the Secretary March 16, 1932.

[27] "Structural Engineering, Fundamental Properties of Materials," p. 97.

In Example 1, commenting on Fig. 2(*c*), the author states parenthetically that, "the ratio, $\frac{T'}{T_1} = \frac{1}{1.04} = 0.96$." For the case given, $0.96 = \cos\alpha$ (the angle of lay); hence, according to the author, the ratio of the stresses in the center wire, and the curved wire equals $\cos\alpha$. As α approaches $\frac{\pi}{2}$, however, $\cos\alpha$ approaches zero, and the ratio of the stresses in the center and curved wires must also approach zero, according to the foregoing hypothesis. This does not appear reasonable.

According to statics,

$$a'\, T' + a_1\, T_1 \cos\alpha = P \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(38)$$

in which, a' is the area of the straight wire; a_1, the area of the curved wire; and P, the total applied load. If α is zero, the wires share the load equally (provided $a' = a_1$ and the elastic properties of the wires are identical); but as α approaches $\frac{\pi}{2}$, the curved wire supports a lesser, and the center wire a greater, proportion of the total load, P.

According to Fig. 2 and Hooke's law, provided the proportional limit is not exceeded:

$$T' = \frac{E'\,(dl')}{l'} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(39)$$

while,

$$T_1 = \frac{E_1\,(dl')\sec\alpha}{l'\sec\alpha} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(40)$$

in which, E' and E_1 equals the moduli of elasticity of the center and curved wires, respectively. According to Equations (39) and (40),

$$\frac{T_1}{T'} = \frac{E_1}{E'} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(41)$$

that is, the stresses in the wires are equal if $E_1 = E'$.

Combining Equations (38) and (41),

$$T' = \frac{P}{a' + a_1 \frac{E_1}{E'} \cos\alpha} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(42)$$

and,

$$T_1 = \frac{P}{a' \frac{E'}{E_1} + a_1 \cos\alpha} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(43)$$

are obtained.

If instead of being surrounded by only a single wire as indicated by Fig. 2, the center wire is surrounded by several wires, Equation (38) becomes,

$$a'\, T' + \Sigma\,(a_1\, T_1 \cos\alpha) = P \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots(44)$$

and Equations (42) and (43) become, respectively,

$$T' = \frac{P}{a' + \Sigma \left(a_1 \dfrac{E_1}{E'} \cos \alpha\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

and,

$$a' \, T_1 \frac{E'}{E_1} + \Sigma \left(a_1 \, T_1 \cos \alpha\right) = P \quad \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

If n_1 equals the number of wires to which T_1, a_1, and a, apply, n_2 equals the number of wires to which T_2, a_2, and β apply, and n_3 equals the number of wires to which T_3, a_3, and λ apply, etc., Equations (44), (45), and (46) may be written in the more general form:

$$a' \, T' + n_1 a_1 \, T_1 \cos \alpha + n_2 a_2 \, T_2 \cos \beta + \ldots. = P \quad \ldots\ldots\ldots (47)$$

and,

$$T' = \frac{P}{a' + n_1 a_1 \dfrac{E_1}{E'} \cos \alpha + n_2 a_2 \dfrac{E_2}{E'} \cos \beta + \ldots.} \quad \ldots\ldots\ldots (48)$$

$$T_1 = \frac{P}{a' \dfrac{E'}{E_1} + n_1 a_1 \cos \alpha + n_2 a_2 \dfrac{E_2}{E_1} \cos \beta + \ldots.} \quad \ldots\ldots\ldots (49)$$

$$T_2 = \frac{P}{a' \dfrac{E'}{E_2} + n_1 a_1 \dfrac{E_1}{E_2} \cos \alpha + n_2 a_2 \cos \beta + \ldots.} \quad \ldots\ldots\ldots (50)$$

and so on.

If P is the limiting load for the strand and Σp the limiting load for all the individual wires which make up the strand (when the wires are straight), the efficiency is,

$$\frac{P}{\Sigma p} = \frac{a' \, T' + n_1 a_1 \, T_1 \cos \alpha + n_2 a_2 \, T_2 \cos \beta + \ldots.}{a' \, T' + n_1 a_1 \, T_1 + n_2 a_2 \, T_2 + \ldots.} \quad \ldots\ldots\ldots (51)$$

Then, for the example cited by the author (that is, a center wire surrounded by several others having the same areas as the center wire, and $\alpha = 15°\text{-}35'$), $\cos \alpha = 0.96$. Equation (51) becomes,

$$\text{Efficiency} = \frac{1 + n_1 \cos \alpha}{1 + n_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (52)$$

If $n_1 = 6$, as is usual under these conditions, the efficiency of the strand according to Equation (52) is 96.6%, which differs by only approximately 0.5% from the value given by the author. However, in the extreme case, in which $n_1 = 1$ and $\alpha = \dfrac{\pi}{2}$, by Equation (52), efficiency $= \dfrac{1 + 1(0)}{1 + 1} = \dfrac{1}{2}$, or 50%; whereas by the author's method (as understood by the writers), efficiency $= \dfrac{(\cos \alpha) \, 2 \, S'}{2 \, S'} = 0$. The results of the two methods are seen to diverge

as α increases. The writers believe that the author is unwarranted in stating that $\frac{dl_1}{dl'} = \frac{T_1}{T'}$.

Regarding "Strength of a 6 × 19 Rope," Equation (1) may be written in the more general form,

$$X + n_Y Y \cos\alpha + n_Z Z \cos\beta + \ldots = P \quad \text{.............. (53)}$$

in which, n_Y and n_Z equal the number of wires making angles of α and β, respectively, with the axis of the rope. It will be noted that, except for notation, Equations (53) and (47) are identical.

It is not clear what the author means by the phrase "if slipping occurs," nor is it clear how "slipping" is taken into account in his calculations for strength efficiency. The effect of any individual wire slipping with respect to the others would appear to be a function of: (*a*) The manner in which the ends of the strand of rope are secured; (*b*) the length of the member; (*c*) the coefficient of friction between the component parts; and (*d*) the applied load.

If the ends of the member are secured by wire-rope sockets properly installed so that each wire can assume its due share of the applied load, and if the member is of considerable length, the writers believe that fair approximations of stresses and efficiency may be obtained from Equations (47) to (51). However, it is obvious that if the ends of the member are secured so that certain individual wires may be taut while others are slack, the problem is not susceptible of calculation. It is also evident that the effect of inevitable errors in socketing will have more influence on short members, such as those ordinarily used in tests, than on members of considerable length, such as those actually used in construction.

The analysis given neglects the effect of:

(1) The wires and strands drawing together as the load is increased.
(2) The end sockets or other fastenings and attachments causing compression at right angles to the axis of the member.
(3) Compression resulting from each wire or strand pushing against its neighbors.
(4) Friction between the individual wires or strands.
(5) The change in angle of lay as the load is increased.

On account of the effect of the foregoing, especially Item (1), the writers believe that the moduli of elasticity of strands and ropes are not susceptible of even approximate determination by calculations. They should be determined from tests and, if practicable, the tests should be made on members of lengths comparable to those used in construction. However, Mr. James F. Howe has given[28] formulas for the determination of the modulus of elasticity of strands and ropes, and it is remarkable to note that the results obtained from his formulas agree well with values determined by tests on pre-stressed members.

As pointed out by the author the modulus of elasticity of strands and ropes is not constant for low stresses. It has been found that both the elastic

[28] *Transactions*, Am. Soc. Mech. Engrs., Vol. 40, 1918, p. 1058 *et seq.*

limit and the modulus of elasticity of such members can be raised materially by subjecting them to a cycle or two of comparatively high stress, and that the primitive elastic properties are apt to be quite misleading. It is believed that, when erected, all such members should be pre-stressed to at least the highest point to which they are likely to be subjected while in service, and that by this simple procedure their modulus of elasticity can be made practically constant throughout the range of working stresses.

The author gives Equations (3), (4), and (5), as if they were derived from the approximate formula for beams, $EI \frac{d^2y}{dx^2} = M$. The writers believe that this is an erroneous view. Consider an element of a straight bar of length, *ds,* and having a radius of cross-section, r'. Let this bar be bent until the radius of the arc is R. Let $d\theta$ equal the angle between two radii subtended by ds. Then, $ds = Rd\theta$, but the unit elongation of an extreme fiber equals $\frac{r'd\theta}{ds} = \frac{b}{E}$; hence, $b = E \frac{r'}{R}$, which is the author's Equation (3). The only assumptions made in deriving this formula are that "plane sections remain plane," and that E is constant and the same for tension as for compression (the stress as well as the strain has plane distribution), which assumptions are substantially true within the proportional limit. The approximate formula, $EI \frac{d^2y}{dx^2} = M$, referred to by the author, is based on the further assumption that the curvature, $\frac{1}{R}$, is small and, therefore, its exact value, $\frac{d^2y}{dx^2}\left(\frac{1}{1+\frac{dy^2}{dx^2}}\right)^{\frac{3}{2}}$, may be taken as equal to $\frac{d^2y}{dx^2}$. This assumption is not involved in the derivation of Equation (3). This equation is not applicable beyond the elastic limit, because then E is not constant. Since the working stress should not exceed the elastic limit the more involved stress conditions prevailing beyond that point are only of academic interest and ordinarily need not be investigated. The well-known Reuleaux formula (Equation (4)) is obtained from Equation (3) by multiplying both numerator and denominator of the right-hand side by 2. The writers fail to see where there is anything "arbitrary" in the argument leading to Equations (3) and (4), and believe them to be rational. From Equation (3) it follows that,

$$b = Er' \, \Delta \frac{1}{R} \quad \text{.........................(54)}$$

in which, $\Delta \frac{1}{R}$ is the change in curvature due to flexure.

It has been shown[29] that for a rope,

$$\Delta \frac{1}{R} = \frac{2 \cos^2\alpha \cos^2\sigma}{D} \quad \text{....................(55)}$$

[29] *The Engineering Review* (Lond.), October, 1908, paper by R. W. Chapman.

in which, α is the angle of lay of a single wire with the axis of a strand, and σ is the angle of lay of a strand with the axis of the rope. Substituting this value of the change of curvature in Equation (54) and substituting d' for $2r'$, Equation (54) becomes[29],

$$b = \frac{Ed' \cos^2\alpha \cos^2\sigma}{D} \qquad (56)$$

It will be noted that Equation (56) differs from Chapman's formula (Equation (12), as given by the author.

Taking cos α and cos σ as each equal to 0.95 and E as 28 500 kips per sq. in., for a $\frac{3}{4}$-in., 6 × 19 rope bent around a 6-in sheave, Equation (56) gives,

$$b = \frac{28\,500\ (0.95)^4 \left(\dfrac{0.75}{15}\right)}{6} = 193.5 \text{ kips per sq. in.}$$

for the extreme fiber stress due to flexure, in the wires bent on the sharpest curvature. Equation (56) does not state that the total tensile stress on the rope due to bending is the product of b and the area of cross-section of the rope; that is, $193.5 \times 0.241 = 46.6$ kips, as intimated by the author. Since the extreme fiber stress obtained is above the proportional limit, and since Equation (56) is based on the hypothesis that this limit be not exceeded, the only true conclusion that can be made from the result — $b = 193.5$ kips per sq. in. — is that the proportional limit has been exceeded. However, assuming that the proportional limit of the wires is greater than 193.5 kips per sq. in. and that the radii of bend of all the wires are equal to the radius of the sheave, then,

$$T = 2 \times 6 \times 19 \frac{b}{r'} \int_0^{r'} \left[(r')^2 - y^2 \right]^{\frac{1}{2}} y \, dy = 76\, b\, (r')^2$$

$$= 76\ (193.5)\ (0.000625) = 9.2 \text{ kips}$$

Chapman made a number of experiments to determine the forces required to bend wire ropes and "found a close agreement between the observed and calculated deflections".[30] His calculations were based on Equation (56). In the concluding paragraph of the paper, in which he reports the results of these experiments, Chapman states that, in applying the formulas to any particular ropes, it must be borne in mind that they were established on the assumption that the wires of the rope are not strained beyond the elastic limit. Furthermore, he points out that it is easy to get ridiculous results from the formulas by applying them to ropes bent over too small pulleys. His calculations were applied only to cases in which the wires are not strained to such an extent that they do not completely recover.

Since friction between the various individual wires is not taken into account in the derivation of Equation (56), it would appear that the extreme fiber stress, b, obtained from this equation would be low rather than high.

[30] *The Engineering Review* (Lond.), October, 1908, p. 264 *et seq.*

If the static friction between the individual wires is sufficient to develop the longitudinal shear required to make the rope act as a beam having a solid cross-section, the following equation,

$$b = E_r \frac{d}{D} \text{ .... } (57)$$

(in which, E is the modulus of elasticity of the rope and d, the diameter of the rope) rather than Chapman's formula, should be applicable. As the ratio of the static friction to this longitudinal shear between the wires approaches zero, the extreme fiber stress due to flexure should approach the values given by Chapman's formula. All cases (for stresses within the proportional limit) lie between these two extremes. Chapman[31] found that the area of the hysteresis loop of the deflection force diagram was materially decreased by lubricating the rope. It would appear, therefore, that b is a function of the degree of lubrication, and since friction can not be entirely eliminated, the value of b is somewhat higher than that given by Equation (56). For a given coefficient of friction, the total friction between the individual wires and strands will be a function of the axial tension, P, on the member and hence, as stated by the author, "the bending effect is a function of the tension" (see "Loss of Strength in Wire Rope Due to Bending"). It is clear, also, that the friction between the individual wires and strands is a function of the degree of tightness to which the individual wires are laid up in the strands, and the strands in the rope.

It has been shown that for parallel wire cables the force required to produce a given deflection is materially effected by the tightness of the wrapping wire.[32] With a tension of 96 lb. in the wrapping wire, it required an applied central load of only slightly more than 4 000 lb. to cause 1.0-in. deflection in a 9-ft. cable containing 1 904 individual No. 6 wires, while with a tension of 200 lb. in the wrapping wire, it required a load of 23 000 lb. to cause the same deflection. In these tests, the cable was mounted so as to act as a pin-ended beam subjected to a concentrated load at mid-span. It would appear that in ropes and strands the outer layers of wires have an effect on the remainder comparable to the effect the wrapping wire of a parallel-wire cable has on the longitudinal wires.

The writers do not find the arguments leading up to Equations (13) to (20) clear. It is believed that the value of the paper would be considerably enhanced, if the author would clarify and amplify these arguments. It should be noted that the well-known equations given for the potential energy of bending and twisting, $U'_b = \frac{M^2_b l}{2\,EI}$ and $U'_t = \frac{M^2_t l_1}{2\,G_r I_p}$, are based on the proportional limit not being exceeded and, therefore, calculations depending on them are only applicable for stresses below that limit.

[31] *The Engineering Review* (Lond.), October, 1908, Fig. 3, and adjacent text, p. 265.

[32] "Construction of Parallel Wire Cables for Suspension Bridges," pub. by John A. Roebling's Sons Co., Chart I and p. 10 *et seq.*

Formulas have been proposed elsewhere [33] for the limiting load on ropes subjected to static and moving loads. These are based on Chapman's findings, the law of conservation of energy, and the obvious fact that if the rope is to last, the limiting stress should be not more than the endurance limit. Of course, ropes used for temporary rigging may be stressed well past this point without serious consequences. However, this does not imply that it is good practice so to stress rope used in permanent engineering structures of any importance.

[3] *Civil Engineering,* March, 1931, p. 547.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

WIND-BRACING CONNECTION EFFICIENCY

Discussion

BY MESSRS. L. E. GRINTER, C. R. YOUNG, C. H. SANDBERG, N. A. RICHARDS, JACOB FELD, DANA YOUNG, AND A. J. WILCOX.

L. E. GRINTER,[33] ASSOC. M. AM. SOC. C. E. (by letter).[33a]—This paper opens for discussion an important division of the general field of structural design. Its publication is timely in that there seems to be an unusually wide interest in the subject of wind-bracing at the moment. The extension of the tall building frame to the 1 000-ft. tower naturally has led engineers to conclude that a review of standard methods of design is desirable.

Mr. Berg undoubtedly has many followers in his belief that the simple approximate methods that have characterized such design, need to be replaced by more exact processes. The writer accepts this viewpoint wherever he is convinced that the resulting complication with its attendant possibility of error can be justified by greater reliability in the finished structure. Unless such increased reliability can be demonstrated beyond reasonable doubt, there is always an important advantage in reducing analysis and design to the simplest possible processes.

The mere fact that hundreds of buildings have survived heavy wind storms, many of hurricane intensity, is a reasonable indication that wind-bracing as evolved in the past cannot be radically wrong, although such observations do not preclude improvement in the future. Where (as Mr. Berg shows) an application of the theory of elasticity indicates stresses twice as great as those calculated by the designer, one may well investigate either the assumptions upon which the analysis is based, or the hypothesis of elasticity.

The author discusses the action of tension rivets through the flange of an I-beam stub connection, and by use of certain assumptions, shows that the tension stress in a rivet may be nearly twice the stress found by dividing the total pull by the number of rivets. His major premise is that the flanges are held flat against the face of the column by the initial tension in the rivets, as shown in Fig. 16(*a*). However, it is important to realize that when

NOTE.—The paper by U. T. Berg, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: April, 1932, by Messrs. David Cushman Coyle, William R. Osgood, O. G. Julian, Harold S. Richmond, and Robins Fleming.

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[33a] Received by the Secretary February 16, 1932.

stressed above their initial tension the rivets will elongate, and the deflected shape of the connection may approximate the condition shown in Fig. 16(*b*) even before any permanent elongation occurs in the rivets. Certainly, only small permanent distortions of the rivets are necessary to permit the action shown in Fig. 16(*b*) and to change the stress per rivet to the total load divided by the number of rivets.

The question then arises as to whether a situation such as this does not justify the neglect in design of the relatively high rivet stresses which Mr. Berg shows may exist at lower loads. The writer would make an important

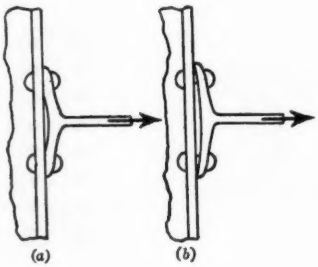

FIG. 16.

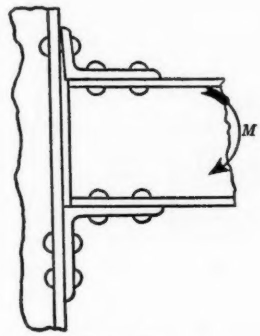

FIG. 17.

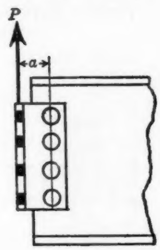

FIG. 18.

distinction between this case and another which is also discussed in the paper; that is, the case of two lines of tension rivets through one leg of an angle. In the latter case, one line of rivets must be stressed far past the yield point in order to stress the second line appreciably, and it seems impossible to justify the neglect of this fact.

The author's discussion of the stresses that exist in the rivets of wind-bracing angles is of particular interest. As in the case of the stub **I**-beam, there is a possible reduction in the tension which he would calculate in the rivets, from the effect of elongation of the rivets after the initial tension has been exceeded (see Fig. 17). When considered from this viewpoint, the advantage of using thick angles (to reduce stress and deflection) becomes apparent. Again, there is a clear argument to support the contention that such high rivet stresses at low loads are of less importance than their values may indicate, since an increase in the load does not increase the stresses in direct proportion.

Mr. Berg discusses standard end connections for beams, and mentions an objection to the use of heavy connection angles, namely, that their stiffness may be sufficient to overstress the tension rivets. His point is well taken, but the discussion is not complete until one has investigated the actual stresses in the rivets through the web of the beam. Standard handbooks give the value of this connection as the direct shear or bearing value of the rivets through the web of the beam. Actually, there is a bending moment to be resisted, and, if the angles can be made sufficiently flexible to reduce the stress in the tension rivets to a low value, the bending moment is approxi-

mately equal to the load, P, times the lever arm, a, as shown in Fig. 18. On the other hand, one then finds that the stresses in the rivets through the web of the beam are increased greatly by the moment of the eccentric force.

As an example, consider the simple end connection shown in Fig. 19. The value of the connection is 11 250 lb., which is the resistance of the rivets in bearing on the web.[34] Considering the moment of the eccentric force, one finds that the maximum stress per rivet is increased from 5 625 lb. to approximately 20 500 lb. Since the working stress in the rivet is only 5 625 lb., the connection is seriously overstressed. Clearly, the tension resistance of the rivets through the outstanding legs of the angles is required to produce a

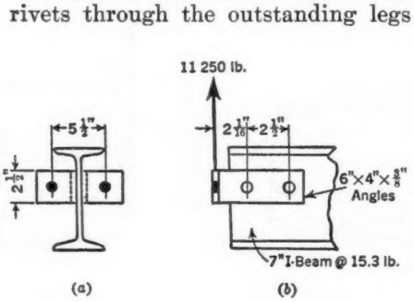

FIG. 19.

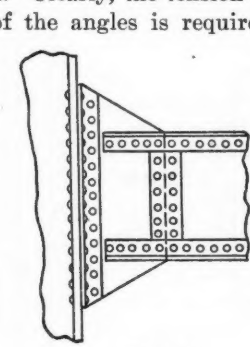

FIG. 20.

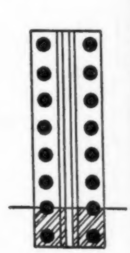

FIG. 21.

resisting moment that will oppose the moment of the eccentric force. If the effective lever arm for this tension is taken as one-half the length of the angle less $\frac{1}{4}$ in., or 1.0 in., the rivet stress to produce a resisting moment equal to the moment of 37 300 in-lb. produced by the eccentric force, would be 36 000 lb. per sq. in. (The rivet area is taken as 0.518 sq. in., for $\frac{3}{4}$-in. rivets.) The stress of 36 000 lb. per sq. in., represents the approximate elastic limit of the rivet.

Since the foregoing resistance is the full moment value that can be obtained from the tension rivets until the connection has been seriously distorted, the stresses in the rivets through the web of the beam will increase much more rapidly than the load for loads greater than those used in design. The use of reasonably stiff connection angles to insure moment resistance, therefore, seems necessary, even if their effect may be to stress the tension rivets highly. To obtain a thickness of angle that would produce a nicety of balance so that the two groups of rivets would approach their elastic limits at the same time, would be desirable, but with standard connections one probably must depend upon some plastic deformation to accomplish essentially the same purpose.

It appears expedient to mention here a misconception that seems common in regard to the proper calculation of the tension stress produced in a line of rivets by flexure. Such rivets occur through the outstanding leg of a clip angle, such as may be used on a wind-bracing gusset (see Fig. 20). Since the stiffness of the gusset in its own plane is extremely high, it is assumed

[34] Handbook of Steel Construction, Am. Inst. of Steel Construction, First Edition (1932), p. 277.

that its connection to a relatively stiff column remains essentially a plane section. The beam formula, $S = \frac{Mc}{I}$, may be used, therefore, to calculate the stresses in the tension rivets, but it is necessary first to locate the neutral axis. When the connection is unstressed by moment, there is a uniform pressure between the clip angles and the face of the column, caused by initial tension in the rivets. Ordinary negative moment in the beam will reduce the bearing pressure at the top of the connection and increase it near the bottom. Until the maximum reduction in pressure equals the initial pressure, the true neutral axis lies outside the section, but the reduction in pressure can be determined correctly by assuming a neutral axis at the mid-height of the vertical angles.

After the initial tension in the upper rivet is overcome by the applied moment, an entirely different condition prevails, and the true neutral axis gradually moves downward from the top toward the bottom of the angles. At failure, the connection undoubtedly 'rides" on the bottom of the angles, the neutral axis having reached its lowest possible position.

During the initial period when the angles are in bearing on the face of the column, the neutral axis for flexure lies at the mid-height, but since the neutral axis actually approaches the bottom of the angles at the point of failure, it seems highly conservative to compute the stress from the first position of the neutral axis. On the other hand, to assume that the neutral axis is at the lower edge of the connection angles actually assumes that failure is imminent, since the upper rivets, being placed perhaps 2 or 3 ft. above the neutral axis, would be severely distorted by even an extremely small rotation. As a reasonably conservative procedure between these two extremes, the writer has been accustomed to take the neutral axis at the center of gravity of the effective section, considering the cross-sectional area of tension rivets above, and the entire area in bearing below, the neutral axis. This area is shown cross-hatched in Fig. 21.

The author discusses the need for considering gravity forces (dead load plus live load) in the design of wind connections. He indicates, without definite recommendation, that wind-bracing connections should be designed for dead load, live load, and wind, although he states that gravity moments at the joints may be reduced as much as 10%, or even more, by elastic distortion of the connection angles. It seems desirable to reconsider the possible effects that may reduce the bending moment as caused in a wind connection by gravity loads. These may be listed as follows:

(1) Elastic distortion of the connection angles;
(2) Slip in the riveted connection;
(3) Elastic elongation of tension rivets when stressed above their initial stress; and
(4) Plastic deformation of the rivets, connection angles, end of beam, or face of column, when the stresses exceed the elastic limit.

Only the first item of this list is considered by Mr. Berg. The second item, slip in riveted connections, has been treated[35] by Professors Wilson and Moore.

[35] *Bulletin No. 104*, Eng. Experiment Station, Univ. of Illinois.

They have demonstrated that a wind connection of the type considered by the author may be expected to show an angle change due to slip of nearly 0.003 radian when the rivets through the beam flanges are stressed to one and one-half times their normal working stress in shear. It is of interest in a study of this effect to note that the end rotation of a simple beam uniformly loaded, where the span is ten times the depth, is approximately 0.004 radian when stressed to a normal working stress. However, a wind connection of the angle type would not be designed capable of developing the moment resistance of the beam, and, hence, the slip alone might readily be sufficient to release practically all end restraint in so far as dead load is concerned.

It is to be noted that the foregoing discussion does not consider end adjustments that may occur as the connection approaches its ultimate resistance. The ultimate strength of the connection tested by Professors Wilson and Moore corresponded to a load of more than two times the load that produced a slip of 0.0028 radian, and the maximum rotation was approximately 0.015 radian.

At several places Mr. Berg mentions the fact that "marked re-adjustments" of moments may be expected to occur when stresses pass the elastic limit. Undoubtedly, re-adjustments do occur, and perhaps the usual tendency is for the re-adjustment to relieve distress in the structure by preventing rapid increase of stress at points of weakness. Nevertheless, one should not depend too seriously upon this phenomenon. Consider a panel of a building in which wind connections are identical at the two ends of the girder. If the point of contraflexure in the girder moves sufficiently toward the right to overstress the left-hand connection so that a partial failure develops, there will probably be an immediate shift of the point of contraflexure to the left-hand side of the center with a resultant over-stressing of the right-hand connection. If this damage is started by a faulty joint, the resistance will then be transferred entirely outside this panel, possibly without serious damage to the structure, but if the over-stress is general throughout several panels of the structure, failure is impending.

Mr. Berg's paper was quite properly restricted to riveted joints. On the other hand, the structural engineer now faces an entirely new problem in this field, the design of welded wind-bracing connections. A service can be performed by any one who will study the design of welded connections from the viewpoint adopted by Mr. Berg. Where welded joints are designed properly, essentially perfect continuity is obtainable, and the design of the joint for both gravity loads and wind loads becomes necessary. An analysis of welded joints by the theory of elasticity would not be open to the criticisms that have been mentioned in this discussion.

C. R. Young,[36] M. Am. Soc. C. E. (by letter).[36a]—The distinctive weaknesses of "knuckle" connections are treated in a definite and quantitative manner in this commendable paper by Mr. Berg. That knuckle connections have proved satisfactory for low buildings is probably due to the fact that the

[36] Prof. of Civ. Eng., Univ. of Toronto, Toronto, Ont., Canada.

[36a] Received by the Secretary February 29, 1932.

force of the wind never reaches the intensity prescribed in the specifications. If, in addition to the negative moments due to gravity loads, there were anything like the expected wind moments, a less satisfactory story of these connections would have to be told.

Recognition of the existence of high stresses in connections due to end moments is not new in structural literature. In discussing web connections of beams to girders or columns, Jacob Friedland[37] pointed out twenty years ago the existence of high tensile stresses in the rivets connecting the beam to the girder or column. For characteristic I-beams, tensile stresses in the rivets, based on the area of the rivet as driven, were found to be as high as 29 100 lb. per sq. in., and on the basis of the area of the rivet before driving, as high as 34 200 lb. per sq. in. There is no doubt that in countless cases the tension rivets, under gravity loads only, have been stressed close to the elastic limit of the material in them, and probably beyond the initial stress in the rivets.

In asserting in the first paragraph that "the mass inertia creates additional resistance against gusts of wind," the author is correct if he means that an addition to mass lessens the frequency; but he is in error if he means that it lessens the deflection or amplitude of vibration that would follow the application of wind force. As has been pointed out by H. V. Spurr,[38] M. Am. Soc. C. E., and D. C. Coyle,[39] M. Am. Soc. C. E., the deflection and amplitude of vibration of a building are independent of mass. This is evident if, for simplicity, the building be considered as a simple cantilever. In the common deflection formula no quantity appears representing the mass of the structure. However, the weight of a building affects the frequency of vibration since, other things being constant, the frequency varies inversely as the square root of the mass.

There is merit in basing the allowable load on a rivet on its initial tension value. While against failure there is the entire margin between actual load and ultimate strength, the increased emphasis now being placed on rigidity makes it particularly desirable, under all circumstances, to keep below the point at which the deformation of the joint will begin to be affected by tension rivet yield. Based on work done at the University of Toronto in 1928, the writer has held that within the initial tensile value of the rivet there is no appreciable elongation.[40] He consequently concurs in the author's assumption to that effect.

The soundness of the choice of representative test values for tension rivets made by Mr. Berg from the results of Professor Wilson and Mr. Oliver may not be entirely obvious. The average ultimate strength of rivets in tension was not 54 000 lb. per sq. in., as might appear from the statement following Equations (2), but considerably greater. This value was the minimum mean observed and was for $\frac{3}{4}$-in. rivets (effective diameter, $\frac{13}{16}$ in.), driven with

[37] *Engineering News*, April 25, 1912, p. 786.

[38] "Wind Bracing", pp. 114–115.

[39] *Civil Engineering*, May, 1931, p. 700.

[40] "Permissible Stresses on Rivets in Tension", by C. R. Young and W. B. Dunbar, *Bulletin* 8, Section 16, School of Eng. Research, Univ. of Toronto.

air-hammer at a cherry-red heat and having a grip of 5 in. For a 2-in. grip the mean was 63 900 lb. per sq. in. In low buildings, and for the upper stories of tall buildings, where the column metal is thin, the grip is likely to be nearer 2 in. than 5 in.

The yield point assumed—that is, 37 000 lb. per sq. in.—is for the material from which the rivets were made. It is not the yield point of the rivet as driven, since that quantity was not observed by Professor Wilson and Mr. Oliver, or in the University of Toronto tests.

As the the initial tensile stress would probably be a smaller fraction of the yield point of the rivet as driven than it is of the bar stock, or the manufactured rivet, an assumed initial tensile stress of 70% of the yield point of the bar is fair.

The author has applied Equation (3) in a manner not intended by its proposers. In the presentation of the results upon which this formula was based,[41] it was recommended that the eccentricity, e, should be taken as about two-thirds the value of the gauge for the outstanding leg. This distance was supposed to give, in typical cases, the position of the point of inflection with respect to the rivet center. As far as the rivet under consideration is concerned, the application of a force parallel to the rivet at the point of inflection would produce the same result as a load applied by a knife-edge at a corresponding distance from, and parallel to, the gauge line. It was in this manner that the specimens tested in combined tension and flexure were loaded, and the formula should be used assuming that the applied force has an eccentricity equal to the distance from the rivet center to the point of inflection.

On this basis the values of f_r from Equation (3) would be as given in Table 5. Examination of this table shows that the two equations compared, give results that are very close for 1-in. rivets, and that Equation (3) gives somewhat higher values than Equation 2(b) for the smaller rivets.

TABLE 5.—Comparison of Working Loads on Rivets Subjected to a Combination of Tension and Flexure

Nominal size of rivet, in inches	Assumed X, in inches	Eccentricity, $e = \frac{X}{0.65}$ as used by author, in inches	Eccentricity, $e = X$, as used by writer, in inches	Permissible Tension per Rivet, in Pounds: Equation 2 (b), gross area	Equation (3), nominal area
¾	0.75	1.15	0.75	3 840	4 520
¾	1.50	2.31	1.50	2 690	3 660
⅞	0.75	1.15	0.75	5 100	5 550
⅞	1.50	2.31	1.50	3 560	4 370
1	0.75	1.15	0.75	6 550	6 480
1	1.50	2.31	1.50	4 680	4 940

Fig. 6 is apparently based on the application of an equation similar to Equation (8), although the author does not so state. Insufficient data are given in the text to permit the checking of the curves. In order to do this it would be necessary to know the values of B and b used by the author. In

[41] *Bulletin No. 8*, Section 16, School of Eng. Research, Univ. of Toronto, p. 408.

establishing Equation (11) for the angular rotation imposed upon the beam by the connection, apart from the effect of tension rivet yield, the author has neglected two factors.

To make this clear, let it be assumed that for purposes of measuring rotation, or tilt, a plane is passed transversely through the beam at the outer edge of the connection and parallel to the face of the column. The movement of this plane away from the column at the tension face of the beam would be influenced (neglecting tension rivet yield) by bending in the tension connection, by elongation of the material of this connection, by shear deformation or slip of the rivets through the beam flange, and by the extension of that part of the beam between its end and the section plane. Similar influences would be operative at the compression face of the beam. The result of including the two factors not considered by Mr. Berg would be to permit a greater rotation than he calculates and a correspondingly smaller moment of restraint.

Another factor not discussed in detail by the author is the effect of repeated and reversed loading. For the rivets attaching the connection to the beam flanges, for example, there would be under stress a deformation which, in part, would be non-elastic and, in part, elastic. The ratio of these will change with the repetition of stress, and what might be called a steady elastic state is not reached until several repetitions or reversals have occurred.

If one were to make a general criticism of Mr. Berg's paper, it would be on the ground that it is rather too greatly condensed. It would have made for clarity if he had indicated his processes a little more in detail. For example, in Table 1 the values of the rivets given in the last three columns are based on areas of rivets as driven and not on the nominal area. In the second paragraph under "Flange Angle Connections," it should be made clear that it is the elastic line of the deformed angle that remains vertical at the rivet, R_1 and horizontal at the rivet, R_2 and not the rivets themselves that preserve these directions.

C. H. Sandberg,[42] Esq. (by letter).[42a]—This paper brings to the attention of the Engineering Profession a greatly neglected phase of wind-bracing design in tall steel buildings. As pointed out by Mr. Berg, since connections of the type shown in Fig. 2 are designed for shear only, on the assumption that the details are so flexible that only a negligible end moment is developed, there is danger that excessive tensile stress will be developed in the rivets unless this phase is specifically investigated. The author's analysis showing the possibility of high tensile stress in this and other types of connections, therefore, is considered timely.

Since 1927 or 1928 there has been an abundance of printed matter on wind-bracing analysis, but very little has been written on the equally important subject of the moment resistance of riveted joints. The few papers that have been published on the subject have been based on assumptions unsupported by experimental data. In the writer's opinion Mr. Berg's

[42] Asst. Engr., Bridge Dept., A. T. & S. F. Ry. System, Chicago, Ill.

[42a] Received by the Secretary March 5, 1932.

paper is the best one that has appeared on this subject; yet his results also are based on unverified assumptions. These seem reasonable enough, but designers have no means of knowing how closely they approach actual conditions. In view of this lack of knowledge on the subject, it is surprising that this important design feature has not been studied experimentally long ago.

In 1928, Professor C. A. Hughes and the writer began a series of tests at the University of Minnesota to determine the resistance of different types of riveted joints. The types of connections selected for this investigation are typical of those commonly used in present-day construction.

The deformation of connection details is not quite as simple as indicated by the author in Figs. 3 and 7, in which they are parallel to the plane of the web of the beam only. In addition to these, there are deformations of the connection details and the column flanges due to bending in a direction normal to the web of the beam. This results in a marked difference in the distribution of stress on the rivets adjacent and remote to the web of the beam or the web of the column. This point has been considered by Sub-Committee 31, Committee on Steel of the Structural Division, of the Society, as is evidenced by the statement, "* * * rivets in the column should be kept in single gauges near the column web." [43] If the rivets considered effective for moment are limited to gauge lines adjacent to the web of the sheared **I**-beam and the web of the column, the safe moment of resistance of this type is limited and is much less than has been attributed to it in many designs.

The author cautions against neglecting the combined effect of wind and gravity loads in determining the moment at the end connections, especially where the rivets are in tension. This point is well taken as there are cases, in which brackets are used for end connections, where the negative moment can be larger than $\frac{WL^2}{12}$, due to the fact that the common assumption of constant section throughout the beam does not hold. This may also be true when the spans are unequal.

Connections have been and are being designed, however, for wind moment only, and there are some justifications for this practice. Bracket connections with tension rivets are usually designed too conservatively. One of the common methods is to assume the rotation of the joint at the center of gravity of the tension rivets (rivets into column), while actually the point of rotation is considerably lower. Beams and columns encased in concrete will also add greatly to the rigidity of the connection. A large part of the wind resistance is usually designed to be taken through the spandrels and, in some cases, one or two additional lines. This arrangement makes it necessary to select the wind-resisting lines of beams for wind loads in all except the upper stories, and the actual moment due to the gravity loads in the end connections is a small part of the total. Another factor in support of ignoring the moment due to gravity load is that if a connection on one

[43] *Proceedings*, Am. Soc. C. E., February, 1932, p. 222.

side of the column is over-stressed the connection on the other side of that column will be under-stressed, and an adjustment will take place. There will also be a distribution through the floor which will cause a stronger joint to relieve a weaker one.

It is common knowledge that since about 1927 the use of the sheared I-beam type of connection has been overdone. This is due largely to the fact that it gives minimum interference with architectural features, and also that some designers undoubtedly do not realize that the bracket connection is the next in effectiveness and economy to diagonal cross-frame bracing. In the design of end connections there are as many ramifications as in the problem of analyzing and distributing the wind moments in the frame of the building, and this cannot be done satisfactorily or in a hurry by an inexperienced designer.

The writer wonders whether it is not being unduly conservative to base the factor of safety for tension in rivets on the initial tension rather than on the yield point. Until the yield point is reached, the deformation of the extreme tension rivets in a connection and the consequent "give" or "slip" of the joint will be so slight as to be practically negligible. Another factor that should be considered in this connection is that the measured values of initial tension are probably too high. Information now available was obtained from tests of shop-driven rivets on comparatively small specimens, on which driving and bucking conditions were ideal, and it is reasonable to expect that the average initial tension of field-driven rivets in large joints will be somewhat less.

Considerable work has been done and much progress made in determining the horizontal loads on structures and in developing rational and relatively simple methods of analysis for frames. It is hoped that further tests of riveted connections will be undertaken so that the design of joints of adequate resistance may be placed on an equally satisfactory basis.

N. A. Richards,[44] M. Am. Soc. C. E. (by letter).[44a]—The action of usual types of angle, channel, and beam-section connections is treated in an interesting and stimulating manner in this paper.

Building laws and conservative practice require that beams and girders be generally proportioned on a simple span basis for dead and live loads, but it is quite obvious that, as heavy wind-bracing connections are added, the members cannot so act, and in wind analyses, they are assumed to be restrained at the ends. The added stresses resulting in the rivets and connections, due to live load and dead load action of the members, should receive more thought and attention, and Mr. Berg's paper should stimulate such study.

One feature that might add interesting further study is the prying action resulting from the pivoting of a beam or girder near the outer edge of the supporting seat, and the corresponding lift at the heel of the upper angle as the member deflects under load.

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[44] Vice-Pres. and Chf. Engr., Purdy & Henderson Co., New York, N. Y.

[44a] Received by the Secretary March 9, 1932.

In reviewing the paper the writer found difficulty in following some of the rather abruptly stated mathematical steps; as, for instance, under "Flange Angle Connections," referring to Fig. 5, $M_4 = \frac{1}{2} M_c$, and below Equations (5) and (6). This is probably due to the rustiness in pure mathematics and mechanics acquired in many years of general practice; but, inasmuch as many interested readers will no doubt find similar difficulty, the author would do well to explain the mathematics of the study with a little less abruptness.

The paper as a whole is a distinct contribution, and is well worth study by those engaged in the design of structures involving connections of the types considered.

JACOB FELD,[45] ASSOC. M. AM. SOC. C. E. (by letter).[45a]—The author gives a timely warning of a frequent error. The design of wind-bracing has become involved in so much mathematical detail that engineers have lost sight of rational and logical carrying out of stress analysis. Too often, a careful analysis of stress is completed and then scrapped because clearance or head-room requirements insisted on by the architect do not allow space for the necessary details.

If the engineer insists on the required space he very often finds himself displaced by another engineer who, according to the architect, is a more efficient designer because he can design wind-bracing without using up more space than can be permitted.

As a result, and possibly also because they do not know any better, some designers believe in an extreme theory of "educated stress methods." In practice, the method consists of summating the resisting moments of all beam-to-column connections on a floor and if such sum is greater than the wind moment of that floor, calling it a sufficient design. If not, a few additional rivets put in here and there make up the necessary total. The method is really based on the principle of least work, or its special statement known as the Moseley principle of least resistance, which is that, if a series of resistances consistent with the conditions of stability can be determined, that series is sufficient to take care of all external forces, although it may not be the most efficient system that can be devised. The application to the foregoing method of wind-bracing design is in error because the major premise is forgotten; namely, that the assumed resistances must obey the conditions of the problem. One condition is that the summation of forces and moments acting on each unit member and connection must vanish.

The universal assumption that a steel beam is simply supported as far as vertical loads are concerned and that the same beam is rigidly connected as far as lateral loads are concerned, is not consistent with the truth or even with logic.

That similar assumptions are sometimes made in reinforced concrete design is true, but the results appear almost immediately. The large reserve of unused strength which exists in steel structures does not exist in rein-

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[45a] Received by the Secretary March 10, 1932.

forced concrete improperly reinforced because of the low tensile strength of plain concrete. When improper assumptions of steel distribution are made, cracks appear, and the cracks are not always caused by poor workmanship, temperature, or shrinkage during setting. Disregard of moments in columns due to unbalanced loads gives cracks in the exterior faces of exterior columns, especially in flat slab buildings; disregard of the two-way action of panels gives cracks in the slabs along girders, unless special reinforcement is provided across the top of the girders; disregard of the rigidity of connection between beams and columns when unequal spans are supported by the same columns, gives cracks in the tops of the beams when full loads occur on the longer spans; and disregard of the torsion in exterior beams always loaded on one side only gives the diagonal cracks visible in so many concrete lintels. These are only a few of the more common errors of details visible in concrete structures. In flexible steel frames similar errors are not visible; yet the effects exist, and the true factors of safety of the steel frames are much lower than is usually assumed. If the basic assumption of wind-bracing design is that connections in a steel frame are rigid, they must certainly be rigid before the wind starts to act, and the author's recommendations are timely and important.

DANA YOUNG,[46] JUN. AM. SOC. C. E. (by letter).[46a]—The analysis of wind-bracing connections given in this paper is important in calling attention to various factors often neglected in design. Theoretical formulas are developed, which indicate that present design practice is seriously in error. The question arises naturally as to which is more nearly correct, theory or practice. This can be settled satisfactorily only by tests. The formulas given are based on various assumptions, the effect of which, while perhaps slight, cannot be definitely determined analytically. As the author states tests are necessary to establish their accuracy.

A number of tests of tension rivets have been made. However, before fully accepting the formulas given in this paper for tension rivets, it is important to consider the assumptions upon which the analysis is based and how these compare with the test conditions.

As shown in Fig. 3, a section of the angle or sheared beam is taken through the rivet, and bending only in the plane of this section is considered; but, since the rivets are not continuous along the length of the member, sections between rivets are not restrained in the same manner as the section considered. The stress conditions will be different for all sections. This has some indeterminate effect on the rivet value.

The assumption that the connection angle or beam is rigidly fixed (that is, that it has a horizontal tangent) at the center line of the rivet is not strictly true. Since both the rivet and the member to which the connection is fastened are elastic there must be rotation at this point.

As the author states, both the distance, C, and the pressure distribution under that part of the connection are uncertain. For accurate results, it is

[46] Washington, D. C.

[46a] Received by the Secretary March 11, 1932.

necessary to make reasonably correct assumptions for these factors. The importance of these assumptions is shown by Equation (2*b*), from which it is seen that any variation in C or its coefficient will affect the value of S in almost direct proportion.

With a flange-angle connection the rivets are also subject to shearing stress due to the end moment. In addition, the rivet may receive shearing stress from the end shear of the beam. These stresses, in combination with the direct tension, produce a principal tensile stress somewhat greater than the direct tensile stress. In some cases this may affect the strength of the rivet appreciably.

It is practically impossible to determine analytically the effect of these various assumptions, either singly or in combination. Designers must rely on tests to establish the accuracy of the formulas.

The tension rivet tests[47] of Professor Wilson and Mr. Oliver, furnish authoritative data in regard to rivets in pure tension, but this condition does not exist for the rivets used in wind-bracing connections. The tests reported[48] by C. R. Young, M. Am. Soc. C. E., are better for this case because they not only include rivets in pure tension, but also rivets with an eccentrically applied load. For this latter condition, 4 by 4-in. clip angles were riveted to a supporting piece and the load was applied through knife-edges bearing on the riveted leg of the angle (the riveted leg was horizontal) at a distance, e, from the center line of the rivet. Part of the tests were with $e = 1\frac{1}{8}$ in. and part with $e = 2\frac{1}{4}$ in. This method of loading is equivalent to establishing the distance, X, used in the author's formulas at $1\frac{1}{8}$ in. and $2\frac{1}{4}$ in., respectively, from the center line of the rivet. In computing Table 3 the author assumes that this eccentricity is equal to $\frac{X}{0.65}$, but from the method of loading with knife-edges there could be little restraint and, therefore, e must equal X.

TABLE 6.—Comparison of Test and Computed Values for Rivets in Tension

Rivet size, in inches	Ultimate Strength of Rivet, in Pounds per Square Inch				
	Pure tension, test value	Tension with Eccentricity = $1\frac{1}{8}$ Inches		Tension with Eccentricity = $2\frac{1}{4}$ Inches	
		Test	Computed	Test	Computed
$\frac{5}{8}$..........	74 400	41 500	35 000	27 400	22 900
$\frac{3}{4}$..........	74 600	36 500	35 100	26 000	23 000
$\frac{7}{8}$..........	63 100	30 300	29 700	21 000	19 500

While the formulas in this paper do not apply to cases beyond the yield point, it is interesting to compare Professor Young's results with values computed from Equation (2*b*), in the same manner that the flexure formula is used for computing the modulus of rupture. In this case $C = 1.5$ in. From Equation (2*b*), for $e = X = 1\frac{1}{8}$ in., the value of S is 0.47 R, and $S = 0.308\ R$ for $e = X = 2\frac{1}{4}$ in. Table 6 shows the ultimate strength of the rivets as

[47] *Bulletin No. 210,* Eng. Experiment Station, Univ. of Illinois.

[48] Address before the Fifth Annual Convention, Inst. of Steel Construction, October, 1927, (Abstract), *Engineering News-Record,* February 2, 1928, p. 188.

found in the tests and the values as computed. Each test value given is the average of six results. For the computed values, R was taken equal to the pure tension strength as given in Column (2) of the table. Everything considered, the computed and experimental values check very closely. The maximum difference is 20%, and the minimum, 2 per cent. However, it must be noted that these tests do not give any data as to the accuracy of Equation (6) for determining X; nor is it necessarily true that tests with angles of other sizes, or with sheared beams, will show equally close agreement.

A flange-angle connection is subject to forces perpendicular to the direction of the pull, S. This is shown in Fig. 22. As a result there is shear on the column rivet and tension on the beam rivet, in addition to the forces

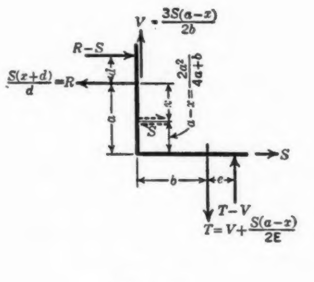

FIG. 22

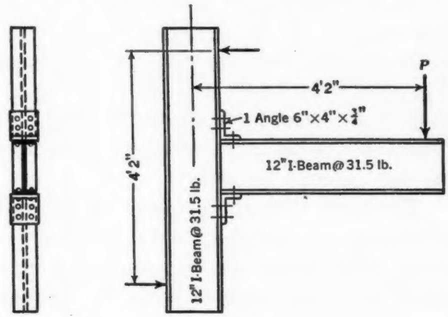

FIG. 23.

generally considered. In most cases these will be small and probably may be neglected. There are also the shearing stresses set up by the end shear of the beam to consider. An idea of the relative magnitudes of the forces involved, may be gained by noting that, in Fig. 22, when $a = b$ and $e = d = \frac{1}{2} a$: $R = 2.2\ S$; $V = 0.6\ S$; $T = 1.0\ S$; and $X = 0.6\ A$.

Equation (12) may be simplified. Since X is a function of A and B, as given by Equation (6), B may be eliminated and Δ_1 expressed as,

$$\Delta_1 = \frac{A^2}{6\,EI}\,(3\,X - A) \qquad (34)$$

For $X = 0.5\ A$, this reduces to $\Delta_1 = \frac{A^3}{12\,EI}$, which is the deflection formula for a beam fixed at both ends, but displaced perpendicularly to its length.

Equation (11) is subject to the various assumptions used in the preceding theory and besides it does not take into consideration the slip in the beam rivets and bending in the column flanges. Rivet slip, especially, will greatly increase the angle of rotation. This may be demonstrated by referring to a test made by Professor W. M. Wilson and H. F. Moore[49] to determine the rigidity of riveted joints. The test piece and method of loading are shown in Fig. 23. For a load, $P = 4\,500$ lb., the rotation at the joint as computed

[49] "Tests to Determine the Rigidity of Riveted Joints of Steel Structures," by Wilbur M. Wilson M. Am. Soc. C. E., and H. F. Moore, *Bulletin No. 104*, Eng. Experiment Station, Univ. of Illinois.

by Equation (11) is $\phi = 0.000156$ radian. The test showed a rotation of $\phi = 0.0028$ radian. This is a large difference and shows the need of further experimentation.

However, regardless of the possible error in the formula for joint rotation, the effects of this rotation, as pointed out by the author, are important. The fact that many buildings have been designed neglecting joint rotation and the effect of gravity moments on wind-bracing connections, does not mean that these factors can always be neglected. The factor of safety, the bracing effect of walls, and the fact that the full design loads are seldom realized, have all helped to cover up poorly designed details.

A. J. Wilcox,[50] M. Am. Soc. C. E. (by letter).[50a]—The design of wind-bracing connections is a subject on which there is a wide difference of opinion although until one has made a considerable study of them, it appears to be a very elementary problem. The author has pointed out some of the conditions that make these connections difficult to design along undisputable theoretical lines. His solution is based on certain assumptions which the writer believes are not entirely logical.

He starts with the assumption that hot driven rivets have an initial stress of at least 70% of the yield point that is set up by the rivet contracting as it cools. The writer has seen many hot driven rivets so free that their looseness could be easily detected with the fingers. If a large number of actual field rivets were tested, they would show a wide variation of stress and the writer would not be surprised if the average of such a test was much less than 70% of the yield point, especially if the rivets were driven without the highest grade of inspection.

Engineering practice of long standing does not permit any such stresses to be used in rivets in tension. The practice, until recent years, was to use the shearing value of 12 000 lb. per sq. in., increased 50% for wind stresses. When this unit stress was increased to 13 500 it was not considered safe to increase the stress for wind 50%, but $33\frac{1}{3}$%, the result being no increase over the 18 000 lb. per sq. in. formerly used. This limit is not definitely set by any building code or standard specifications, but as the result of many years of experience by the leading men in the profession. Many of them were keenly interested in producing the most economical structural steel to meet competition from other forms of construction. This experience must have convinced them that it was not safe to take advantage of an apparent defect in the codes.

The point is often argued that since it is impossible to drive hot rivets consistently without occasionally setting up cooling stresses much in excess of the standard limits, and since it is impractical to relieve them, it is entirely proper to make use of such stresses. However, after considerable designing experience and study of the subject of distribution of wind stresses, the writer feels that the day is far distant when these stresses can be dis-

[50] Structural Engr. (H. G. Balcom), New York, N. Y.

[50a] Received by the Secretary March 15, 1932.

tributed accurately. The more general methods used by many engineers, because of their simplicity, appear to give results that are not approximately correct. Furthermore, the common practice of to-day is to neglect the effect of the continuity of the dead and live load moment. These factors would most certainly alter the designed stress in wind-bracing connections.

The author used the diameter of the hole and not the nominal diameter of the rivets in determining his rivet areas; again, this is against standard practice. This assumes that when separately punched pieces are fitted together, the holes match exactly. Such workmanship requires machine-shop practice. Because of the cost involved in the use of holes punched so accurately that the rivets or bolts would have a tight fit throughout, larger holes are a more practical solution. This assumption requires the rivet to fill the hole completely. With a combination of good detailing, workmanship, and inspection, the writer believes this result can be approached. However, when the grip is long, it is more uncertain. Practically all codes and standard specifications further reduce the standard stress on the nominal area for long-grip rivets. The designer should also be governed on important work by the quality of workmanship he may expect.

Inspection of material and workmanship requires as much skill as design, although along a somewhat different line. The chief inspector and his assistants could be selected with as much care as the designing force. No workman in any trade cares to be made to do his work over again, and many workmen are more skilled in covering up defective work than inexperienced inspectors are in detecting it.

The author also assumes the connecting member fixed at the rivet as shown in his Fig. 3(*c*). He arrives at this by assuming for the distance, C, that there is no deformation over the distance, X. In his Example 1, $C = 1.5$ in. and $X = 0.415\ A = 0.87$ in. If there is a deformation in the length, X, there most certainly is a deformation in the length, C, which is almost twice the value of X. The writer believes an eccentric force must be applied to the rivet, and its amount depends on the stiffness of the connection.

In Example 2, Mr. Berg uses an angle 4 in. long and does not deduct the rivet hole, although he assumes the maximum stress at that point. Besides using the gross length, he assumes a uniform stress over the full length. With moment arms of about 1 in., there is considerable question concerning the proper length to use.

Going further into the design, the members to which the connection is made, should be considered. Four rivets in a horizontal line can be used in a 14-in. column in either web or flange. If moment arms of less than 1 in. are used to design a connection, certainly the outside rivets in such connections are not stressed the same as the inside pair. In investigating the strength of the column at the point of the connection, through what vertical length does the stress act in the flange or web and with what variation of stress?

The principal point, the writer wishes to bring out is that the application of theory to wind-bracing connections is a complicated problem. This also applies to ordinary shear connections. No liberty should be taken with rivet

values and reasonable efforts should be made to favor rivets in tension until some time as sufficient tests have been made, and made public, to indicate to the designer just what distribution of stresses takes place.

Until such time, the single shearing unit stresses should be used for tension in rivets; the connections should be designed using a thickness of metal equal to the nominal diameter of the rivet; and the gauge should be the minimum driving clearance, riding the fillet $\frac{1}{8}$ in. There is also doubt in the efficiency of channel stubs because of their thin webs. This recommendation is based on considerable study, and the results of such meager tests as the writer has seen.

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DISCUSSIONS

PUBLIC SUPERVISION OF DAMS
A SYMPOSIUM

Discussion

BY MESSRS. I. C. STEELE AND WALTER DREYER, FRED A. NOETZLI, GEORGE N. CARTER, GEORGE W. HAWLEY AND H. K. BARROWS

I. C. STEELE,[16] and WALTER DREYER,[17] MEMBERS, AM. SOC. C. E. (by letter).[17a]—The essential and desirable features that should be included in legislation providing for State supervision of the design and construction of dams, has been clearly outlined by Mr. Markwart. The writers are in accord with all that is stated under the headings "Extent of Supervision," "Exemption from Supervision," "Supervision Personnel," "Burden of Cost," "Appeal Provision," "Existing Dams," and "Maintenance of Dams." It is extremely undesirable to incorporate a building code as part of the law because of the continual change in the art of dam construction, and because of the difficulty of making legislative revisions. The adoption of a minimum requirement code as a regulation of the supervising body might not be objectionable, since such a code can be changed from time to time without passing through the State Legislature. The danger then would be that after some years of operation under a tentative code a desire would develop to place the code itself on the statutes, in which case progress in the art would stagnate more or less until a revision was adopted.

Prior to August 14, 1929, dams erected in California by privately owned public utilities were subject to the approval of the State Railroad Commission. This Commission had no jurisdiction over the so-called "publically-owned" utilities. Dams erected by private agencies other than utilities were subject to the approval of the State Engineer. Federal authority exercised

NOTE.—The Symposium which includes the paper by A. H. Markwart, M. Am. Soc. C. E., presented at the meeting of the Power Division, New York, N. Y., January 16, 1930, and the paper by M. C. Hinderlider, M. Am. Soc. C. E., presented at the Technical Session, Sacramento, Calif., April 23, 1930, respectively, was published in January, 1932, *Proceedings*. Discussion of the Symposium has appeared in *Proceedings* as follows: March, 1932, by H. deB. Parsons, M. Am. Soc. C. E.; and April, 1932, by Messrs. William P. Creager, M. M. O'Shaughnessy, N. A. Eckart, R. C. Johnson, F. W. Hanna, and Joel D. Justin.

[16] Chf., Div. of Civ. Eng., Pacific Gas & Elec. Co., San Francisco, Calif.

[17] Asst. Chf., Div. of Civ. Eng., Pacific Gas & Elec. Co., San Francisco, Calif.

[17a] Received by the Secretary February 15, 1932.

supervision over débris dams, dams owned by the Government, and those occupying Government rights of way. Municipalities or quasi-municipalities with regularly organized engineering departments were not subject to State supervision. When no such department existed the dams were subject to the approval of the State Engineer.

Public opinion folllowing the failure of the St. Francis Dam directed the attention of the State legislative body to the necessity for concentrating the supervision of dams under one central control. As a result, the 1929 Legislature, under the police powers of the State and for the purpose of safeguarding life and property, invested the duty of the supervision of the construction and maintenance of all dams, with the exception of those owned by the United States, in the Department of Public Works to be administered by the State Engineer.[18] This law repeals all other State Acts governing the supervision of dams. Its principal features are, as follows:

(1) Dams under jurisdiction are only those structures which are either 15 ft., or more, in height measured from ground level to the spillway, or which impound 10 acre-ft. or more.

(2) The law invests the Department of Public Works with authority under the police power of the State and directs it to supervise the construction, enlargement, alteration, repair, maintenance, operation, and removal of dams for the protection of life and property. This includes all dams and provides for co-operation between State and Federal authorities when both have jurisdiction. In controversy between State and Federal authorities, the Federal jurisdiction predominates.

(3) Owners of existing dams are required to obtain a certificate of approval and all structures must be examined by the State within a period of three years from the date of the Act. Costs of such examination are borne by the State.

(4) An application, accompanied by complete engineering data and a filing fee, must be presented before a new dam can be constructed or an existing dam enlarged. Certain penalties are provided in the nature of additional fees when the actual cost exceeds the estimated cost by more than 10 per cent.

(5) An application must be filed and written approval secured before commencing the repair, alteration, or removal of a dam. When repairs are of an emergency nature, work may be started at once, but the Department must be notified and the work made to conform to such orders as the Department shall issue.

(6) No application will be approved in less than 10 days from the date of filing, and the Department must be given 10 days' notice before construction is begun. Approval becomes void after one year unless the time is extended for cause.

(7) During construction the Department may order the owner to change his plans or methods of construction if necessary to insure the safety of the structure.

[18] Senate Bill No. 723, Chapter 766, Section 1, Statutes of 1929.

(8) The Department is empowered to supervise the maintenance of dams and to take any remedial means it deems necessary in an emergency.

(9) The Department acts on complaints alleging that person or property of complainant is endangered by the dam. The complainant must deposit cash when an inspection is required. If the complainant is sustained, the money is returned, otherwise it is paid into the State Treasury.

(10) No action can be taken against the State due to failure or to the operation of a dam.

(11) The owner is not relieved of any legal obligations or liabilities.

(12) Persons wilfully obstructing the Department are guilty of misdemeanor. If work is knowingly done without an approval, or in violation of an order, the owner or the contractor is also guilty of a misdemeanor.

(13) The law provides for a mandamus or injunction when the orders of the Department are being violated or violation is threatened. Action is brought in the Superior Court of the county in which the work is being done; the owner must answer the petition within twenty days, and after he has answered, or defaulted in answering, the Court investigates and either dismisses the action, makes the injunction or writ of mandamus permanent, or so modifies it as to afford appropriate relief.

It is to be noted that the law includes most of the characteristics desirable in this type of legislation. The only dams that are exempt from supervision are those owned by the Federal Government, and this is due to the superiority of Federal over State jurisdiction. Existing governmental machinery is used to administer the duties created by this Act. Supervision has been properly limited to that necessary to safeguard life and property. The rights of the owner to select the site, to design the dam, to superintend the construction, and to maintain the structure, are in no way impaired, as long as the dam is deemed safe by the State authorities.

The State is empowered to act in an emergency, but this should be unnecessary where the owners are in a financial position to take action themselves. The regulations issued by the Department under the Act require the formal approval of plans before construction can proceed, and do not delineate the procedure for changes initiated by the owner after the work has been started. It is probable that it will be difficult to give formal approval to a set of plans and then have construction conform exactly to those plans. It is usually necessary to make changes as construction proceeds, and it is quite likely that the Department will find it advisable to permit construction to proceed along lines which, in general, follow the original plans, and, at the same time, give the owner latitude to make such changes as he finds advisable. With close and co-operative contact between the owner and the Department, there should be no difficulty in administering this part of the work.

The law properly specifies that the State shall pay the cost of examining existing structures. However, it provides for the payment of fees for the construction of new dams, or for the enlargement of existing dams, that apparently place the burden of the cost of supervision by the State upon the

owner of the dam. This is evident when one considers the magnitude of the fees; for example:

Cost of dam	Fee	Cost of dam	Fee
Minimum	$20	$1 000 000	$5 500
$100 000	1 000	5 000 000	9 500
500 000	3 000	10 000 000	12 500

Appeal Provision.—The Act provides that any owner shall have such recourse to the Courts as he may be entitled to under the laws of the State. When requested in writing to do so by the owner, the Department appoints a consulting board of two, or more, which reports to the Department on the safety features involved. The cost and expense of such a board, if appointed on the request of an owner, are paid by the owner.

While the language of the Act does not make the decision of the Consulting Board binding on the State Engineer, it may be presumed that he would only appoint such a board if he were willing to abide by, or at least be influenced by, its decision or report. If the owner cannot find relief from what he may consider unwarranted interference or unnecessary expenditures through the medium of the Consulting Board, he must seek relief in the Courts. Recourse to the Courts will greatly hamper the progress of construction, and it is unlikely that an owner will adopt this means of relief unless the burden imposed on him by the Department is, in his opinion, unbearable.

It is evident from the foregoing analysis that the new California statute, made necessary by public opinion following the St. Francis Dam disaster, has in general been wisely prepared and contains many of the desirable features that should be included in such a law. While the burden of the cost of supervision has apparently been placed on the owner, after all it is the owner who creates the risk to life and property, and the objection to this means of providing the funds necessary to insure safe construction and operation may be subject to argument. Provision has been made, rather loosely it must be admitted, for relief for the owner in the event of a difference of opinion, and this feature might have been more clearly outlined in the terms of the Act. It would seem reasonable to have the decision of a consulting board appointed by the State Engineer at the request of the owner, binding on both parties. Presumably, however, this can be done by agreement between both parties prior to the appointment of such a board.

In all other respects, the California law seems to be entirely satisfactory to the owners of dams, as well as to the public; and if wisely and judicially administered, with both parties willing to co-operate to the interests of the public safety, its operation should be smooth with few, if any, controversies of importance.

FRED A. NOETZLI,[19] M. AM. SOC. C. E. (by letter).[19a]—Structural engineers have realized for a long time the chaos that would exist without a code for the design and construction of buildings. Dam engineers are slowly

[19] Cons. Hydr. Engr., Los Angeles, Calif.

[19a] Received by the Secretary March 7, 1932.

approaching the point of recognizing the necessity of a code for dams. The excellent papers by Mr. Markwart and Mr. Hinderlider are certainly steps in the right direction.

Table 1, presented by Mr. Hinderlider, records the failures or partial failures of 253 dams in the United States, and of 40 dams in all European and other foreign countries. This record would be rather appalling if it did not include, as properly pointed out by Mr. Hinderlider, so-called "partial" failures which merely required some repair work to put the dams in operating condition again. A number of foreign countries, notably France, Germany, and Italy, have adopted rules and regulations for the design, construction, and maintenance of dams. These rules are sufficiently strict to preclude unsafe designs or constructions. On the other hand, they are sufficiently flexible so as not to restrict progress in the art unduly.

The writer is informed that Arizona is the first State in this country to take steps to adopt a definite code for dams. This code was compiled by J. A. Fraps, Assoc. M. Am. Soc. C. E. California enacted a statute in 1929 placing jurisdiction over all dams in the State, except those of the Federal Government, in the State Engineer. In California, an applicant is required to submit the plans and specifications for a proposed dam to the State Engineer for approval, and to pay a certain fee, depending on the estimated cost of the dam. This fee is intended to cover the State's expenses for passing on the plans and specifications of the dam; further, for the geologic and engineering investigation of the dam site, and for occasional supervision during the construction of the dam. If the applicant should desire to submit the plans for an alternative type of dam at the same site, a second fee of the full amount must be paid, although a relatively small amount of work is involved in passing on a second set of plans.

It may also happen that at a certain site the foundation conditions as revealed by the excavations indicate the desirability of a change of the type, for instance, from a massive concrete dam to a type that is more flexible and less subject to uplift. If such a change should be made a new filing fee in the full amount would have to be paid.

At certain sites the type of dam best suited for the conditions can be determined only by comparative designs. Actual costs of construction are best ascertained by bids from responsible contractors. In the interest of economy and progress, therefore, the asking for bids on alternative designs approved by the State Engineer should be facilitated by requiring only a relatively small additional filing fee for such designs.

It is interesting to note that according to Mr. Hinderlider the State Engineer in Colorado follows the practice of suggesting to applicants for permits for dams the type of structure indicated by the site and geological formation. Inasmuch as dams of different types may be feasible at certain sites—such as, for instance, a gravity dam, an arch dam, a cellular type of dam, or a structure of one of the buttressed types—the problem is decidedly one of economics. While the cheapest type may not be the one best suited

for given conditions, in all cases, there is no good reason why the most expensive type, namely, the gravity dam, should receive first and sometimes sole consideration.

GEORGE N. CARTER,[20] M. AM. SOC. C. E. (by letter).[20a]—It appears that some agency, clothed with sufficient authority, is necessary as a watchman over lives and property situated below dams impounding quantities of water, that would be dangerous if suddenly unloosed. The State is better adapted to perform this function than any other agency. The ruling hand is thus sufficiently local in its nature to acquire first-hand information; yet it is general enough in character to afford some uniformity. By placing this duty with the State, there is assurance of responsibility, because it is definitely placed by legislative enactment. It would be preferable if every owner of a potentially destructive dam could be charged with responsibility for its safe design, construction, maintenance, and operation; but this is not possible. In the interest of public welfare, therefore, a definite and fixed responsibility should lodge some place and the State Government seems the most logical agency for such a protective measure.

No corporation, company, individual, or any other agency wants to build and maintain a dam that is a hazard to life and property. Whenever the supervision of the design, construction, and operation of a dam, which may create potential danger, has been approached from the standpoint of co-operation, rather than as a police function, considerable progress has been made in establishing harmonious relations between the owner of such a structure and the public official whose duty it is to safeguard equally well the interests of all. Generally, it is a sound premise that only through ignorance will an unsafe dam be built or operated. In most instances, if failure occurs, the major loss falls upon the owner; therefore, only in a case of bad faith or inclination to minimize the cost of a dam (which the ownership of, and responsibility for, will pass in title immediately upon completion of construction), could it logically be argued that a hazardous dam was being wilfully maintained. Thus, the majority of cases of questionable safety will fall into the category of ignorance. Others may arise through honest differences of opinion.

Where the shortcoming is the result of ignorance, there is little to be said. Fortunately, these cases generally involve dams or water storage of small magnitude and, by the same token, often limited financial resources and credit. To impose a comparatively large repair or reconstruction charge arbitrarily upon such an owner frequently works a hardship. The owner is beset with two difficulties, he may lose the structure and may also become liable for damages, or he must suffer a financial outlay for improvement he can ill afford. In such cases the public official charged with the duty of correcting the condition has a delicate problem to adjust the requisite repair measures to the owner's financial ability. True, the necessary construction or reconstruction thus thrust upon the owner is a kindness and a favor, but

[20] Civ. Engr., Boise, Idaho.

[20a] Received by the Secretary March 21, 1932.

the public officer doing this must have, and keep, a fine sense of balance in values and not become imbued with a non-governable zeal to do a surpassingly fine job regardless of the cost burden that may be imposed. In some cases, enthusiasm may be the cause of unjustified expense, and in others, inexperience may prevent the application of more economical but equally adequate safety measures. If the public officer, saddled with the responsibility of preserving safety, is over-cautious in his purpose and does not know that the function of an engineer is "to do with one dollar what any one can do with two or three," he may easily go far beyond the exigency and possibly be the cause of virtual confiscation of the property.

If the alleged defect in a dam comes down to differences of opinion between the owner's engineers and those of the State as to adequacy of foundations, sufficiency in design, thoroughness of construction, or existing operating hazard, then there is nothing to do but "thresh out" the matter, and, failing in agreement, call in more and, if possible, superior engineering talent.

The extent or amount of statutory direction for dam supervision that will give the best results is debatable. Some States have seen fit to attempt almost completely to define, by statute, the principles of location, design, and construction of dams; others have enacted only a broad general law, authorizing scrutiny and requiring approval of locations and design that shall be safe. The best results are obtainable by the broader method. The subject of suitable foundations and design of dams is such a broad one, covering as it does so many different conditions, no two of which are often alike, that the formulation of complete statutory rules is almost impossible. It would indeed be a keen and far-sighted engineer or group of engineers who could write a dam code to cover every condition that might be met in the future. The many arguments and controversies always current over the principles of dam design, illustrate the futility of ever writing a code not subject to dispute, and the action that would result from the efforts of a board of engineers would be a compromise in which, possibly, the best judgment of any one would not be expressed. Furthermore, the officer supervising such work can function more efficiently if he is not circumscribed by some statute which, while not applicable, still governs. If the State Engineer has not the ability and judgment to act wisely in each case before him, neither will he be capable, in all probability, of interpreting and following a complex code.

The State of Idaho has a law covering this matter, enacted in 1895. It is short and adequate, the only two objects stressed being safety to life and property, and the authorization granted the State Engineer to do whatever is necessary to provide or restore safety. In case of existing danger an officer of the peace may enforce his orders, if necessary, and the right of appeal to a Court is provided.

Administration of this law has produced a record of every dam in the State that may be potentially dangerous and this record is readily available for any emegency. Its value, however, would be considerably enhanced by filing therewith record drawings made after completion of construction,

showing in all instances departures from the original plans. Cases have developed in which the plan at hand did not conform to the structure as built, and some delay was occasioned thereby when time was precious. A dam failure of any serious consequence has never occurred in the State and Idaho has many dams of size and importance.

Mr. Hinderlider states that the practice in Colorado is for the State Engineer to suggest the type of structure indicated by physical conditions. It is doubtful if this is a function of a State Engineer or other public official charged with supervision. He should only approve or disapprove the plans and specifications laid before him; possibly after disapproval he might appropriately suggest what would be acceptable.

Supervision and control of dams will be accomplished with the greatest satisfaction to a State as a whole when the function is exercised with the least notice and interference, commensurate with absolute safety to life and property.

George W. Hawley,[21] M. Am. Soc. C. E. (by letter).[21a]—In 1929, the State of California enacted what is perhaps the most complete and far-reaching legislation governing the supervision of dams, which has been effected by any governmental agency for a similar purpose. Prior to this legislative enactment there existed in this State, as in many other States, several regulatory agencies with varying degrees of authority and responsibility. These agencies exercised jurisdiction in differing degrees from complete control over some dams to partial, ineffective, divided, or no supervision over others. Lack of centralized authority and responsibility could result only in confusion of authority, divided responsibilities, ineffective supervision, and a condition of insecurity in the minds of interested parties.

The continued economic growth and prosperity of California, in common with that of any semi-arid region having variable stream flow, is in large part dependent on complete economic utilization of its water resources and the degree of flood protection afforded. The natural stream flow of California will admit of very limited increase over present use and, consequently, the future water supply will be obtained chiefly by storage. This storage can be developed only by the construction of dams, which is indicative that as time progresses dam building will be materially extended both in size and number. Since the most favorable sites, topographically, geologically, and economically, are first in order to be developed, it follows that, as requirements for storage and flood control increases, the suitability of the available sites will be less favorable. As a complement of this proposition, the property values and number of lives that might be jeopardized through the failure of any dam, or affected by the construction of a dam, are becoming increasingly great.

There are at present about 800 dams in California, many of which are structures of outstanding magnitude and importance. In many instances dams are constructed at strategic locations as regards potential hazard to populous and rich communities in the event of partial or total failure. When

[21] Deputy State Engr. in Chg. of Dams, Sacramento, Calif.

[21a] Received by the Secretary March 28, 1932.

consideration is given to the fact that the size and number of dams are of necessity materially increasing, that the property values and lives which would be involved in the event of failure of a dam are becoming increasingly greater, and that inherent public fear and apprehension attaches to dams more, perhaps, than to other engineering works, centralized, authoritative, and competent supervision of dams becomes imperative.

Public fear in California has been accentuated by two major failures, involving loss of life, together with the question of safety and integrity that has been raised as to other dams.

Accordingly, in response to widespread public demand for more comprehensive, adequate, and thorough protection from the recurrence of loss attendant upon these failures, the California statute govering the supervision of dams (which it is held is a sovereign duty of the State), was enacted to safeguard the life and property of its people. The constitutionality of the California law has been upheld by two Appellate Court decisions and one Supreme Court decision.

The California law vests authority over all dams in California (other than those Federally owned) that have a capacity of 10 acre-ft. or more, or a height of 15 ft. or more—regardless of ownership or supervisory control—exclusively in the State Engineer. The law was carefully worked out in conjunction with the owners and builders of dams and is believed to be the most complete and effective statute of any State on this subject. The State control is limited to that necessary to safeguard life and property and embraces three main features:

1.—An examination into the safety of all existing dams and their approval for use if found safe or after such repairs as the State Engineer may find necessary are made;

2.—Approval of plans and inspection of construction of new dams, followed by their approval for use; and,

3.—Supervision of maintenance and operation of all dams in so far as necessary in the interests of safety.

Provision is made in the Act whereby the State Engineer is authorized to co-operate with agencies having joint jurisdiction, such as the California Débris Commission, Federal Power Commission, and U. S. Forest Service.

The State Engineer is invested with authority under the police power of the State and directed to supervise the construction, enlargement, alteration, repair, maintenance, operation, and removal of dams for the protection of life and property. All dams in the State whether heretofore or hereafter built or under construction at the date the legislation was effected are under the jurisdiction of the State Engineer and application for their approval must be filed.

Every owner of a dam completed prior to the effective date of the Act is required to file an application for approval of the dam, this application to be accompanied by such available and appropriate information concerning the dam as may be required by the Department.

Subsequent to the effective date of the Act, construction of any new dam or the enlargement, repair, or alteration of any dam shall not be commenced until the owner has applied for and obtained from the Department written

approval of the plans and specifications. It is required that the application for approval of plans and specifications for a new dam shall set forth the location, type, size, and height of proposed dam and appurtenant works; contemplated use and storage capacity of the reservoir; and such other pertinent data as the Department may require concerning foundation conditions, drainage-basin area, precipitation, flood flow, and other appropriate data. It is also required that a filing fee, based on the estimated cost, shall accompany the application. The State Engineer is empowered to approve or disapprove the application or require the modification or revision of any incomplete, defective, or insufficient application.

The action of the Department is restricted to approval or disapproval of the application submitted by the owner and to assurance of proper execution of the work in accordance with the approved plans and specifications. If an application is disapproved the reasons for so doing are set forth. In the case of one major dam, the only dam for which application has been denied to date, the State Engineer disapproved the application on the grounds that the dam, if built as proposed in the application, would be unsafe and a serious menace to life and property in the populous valley below.

During the construction, enlargement, repair, or alteration of any dam the Department is required to make or cause to be made such continuous or periodical inspections, investigations, or examinations as may be necessary to secure conformity with approved plans and specifications; and in order to insure safety, the State Engineer has authority to order revisions or modifications in the plans and specifications, or, if conditions are revealed which will not permit the construction of a safe dam, the approval may be refused or revoked.

To accomplish the desired objective, namely, the determination and establishment of safety of each of 800 existing dams, in addition to supervising new construction, an experienced personnel has been organized to cope with the many involved technical and practical problems.

The personnel and activities of the Department, of necessity, must permit of extreme flexibility to meet the wide field of activities to which it is necessary that they apply themselves, in order to conclude a sound and proper determination of safety. This variation in duties results from the facts that:

1.—The art of design and construction of dams which is in the evolutionary stage is highly technical and advancing with rapid strides;
2.—There is a wide range in type and location of structure;
3.—The variation in geological formations throughout the State makes the study of foundation conditions of each dam an independent, but important problem; and
4.—The volume of work involved in reviewing in excess of 800 dams within the time limit allowed necessitates efficient and competent prosecution of the various activities.

The activities of the Department have been directed along six general lines of endeavor: (1) Field examination and investigation of existing dams; (2) geological examination and report on dam sites; (3) analytical study of the structural features of all dams and appurtenant works; (4) hydrographic study of the drainage-basin area tributary to the dam; (5)

hydraulic study of the spillway of each dam; and (6) competent and regular inspections of all new construction and repair work.

As a result of investigations of the physical conditions of existing dams it has been possible to segregate these dams, for purposes of expediting the work, into three general classes, namely, (*a*) dams needing repair; (*b*) dams requiring further study; and (*c*) dams ready for approval. Approximately one-third of the total constructed prior to the effective date of the Act falls in each class. The action taken in each of the three classes my be summarized, as follows.

Dams Needing Repair.—Owners are informed in what respect their dams are in need of repairs and are requested to take whatever action is necessary to place them in a satisfactory condition. Plans for the necessary repairs are prepared by the owner and submitted to the State Engineer with an application for approval. After approval of the application the repairs are made by the owner under the supervision of the State Engineer's Office. The Department is meeting with a ready response and has received more than 200 applications for such repair to date (1932).

Dams Requiring Further Study.—There are numerous dams concerning which there are insufficient data to base a decision as to safety. The plans may be too meager in detail to make a complete analysis, in which case actual measurements are made. The materials placed in a dam, or the geology of the foundation and abutments, may be questionable, in which case test borings or other means of examination must be made of the dam or site. If the first inspection was made at a low stage of the reservoir another may be necessary when the reservoir is full in order to determine the amount of leakage. Examination into the safety of these dams, must, by law, be completed prior to August, 1932.

Dams Ready for Approval.—Formal certificates of approval are being prepared for those dams for which the investigations have been completed and which in the judgment of the Engineer Department are ready for approval. That a dam has been approved once does not relieve the owner or the State from constant vigilance to assure proper maintenance. Supervision over maintenance and operation is exercised at all times. Applications for repair or enlargement of a dam, even if the dam has been previously approved, must be approved by the State Engineer. A new certificate of approval will be issued upon the completion of the subsequent work.

In dams of magnitude, or where the technical features involved are such as to require it, or when major controversial technical issues are involved, the State Engineer avails himself of the services and advice of experienced consultants especially qualified in the particular phase under consideration to examine into and report upon the safety matters involved, in order that a proper and sound solution of the problem, from a safety viewpoint, may be reached.

It is the aim and endeavor of the Department, rigidly adhered to, to require that the personnel refrain from forming conclusions on the basis of local or prejudiced influence; refrain from imposing unwarranted or dictatorial conditions beyond the requirements of safety; refrain from exerting

unnecessary or onerous influence over construction; refrain from assuming engineering direction; and refrain from expressing opinions or influencing decisions on economic considerations, such as choice of type, location, etc. In other words, the feasibility and economics of any proposed construction rests solely within the discretion of the owner and wholly without the scope of the State Engineer's jurisdiction. On the other hand the personnel stands at all times willing to discuss informally with the engineer (acting for the applicant) controversal problems of design or construction relating to the dam under consideration to the end that the dam, when built, will be a safe structure.

The mandatory duties placed upon the State Engineer in exercising jurisdiction over the dams in this State and the grave responsibility exacted of the State Engineer in the interests of safety merits the co-operation of the owners of dams, engineers, geologists and the public alike. It is most gratifying to find that, to date, the State Engineer of California has enjoyed a splendid spirit of co-operation, willingly and generously given by all interested parties.

To a limited extent only, has approval of a proposed structure been sought, prior to filing formal application with the Department, for the announced purpose of financing, consummating contracts, selection of design, avoiding payment of professional fees, or shifting responsibility. The Department is not willing nor can it lawfully accede to such requests.

The Department, in so far as may be possible and advisable, is endeavoring to program its methods and policies to the end that uniformity of procedure in accordance with accepted engineering practice may be obtained. Only in the event a sound and workable code for the design and construction of dams can be devised and agreed upon by engineers will the Department be willing to formulate prescribed design criteria for the determination of the safety of every dam which, after all, is a problem for individual study. In the interim it seems desirable that concerted action be taken by engineers to establish minimum requirements for design purposes, such as allowable unit stresses, length of path of percolation, slopes of earth and rock dams, uplift pressures, etc.

An analysis of records discloses that a major number of dam failures has resulted from inadequate spillway provisions and foundation or abutment insufficiencies. Where ordinary care has been exercised in design and construction instances of failure within the structure itself have been extremely rare. This indicates not that less thought and effort should be directed to technical design, but rather that more thought should be applied to geologic and hydraulic features.

State supervision of dams, if properly and competently administered, and if directed aggressively to the proper requirements of safety for dam design and construction, merits the support of the Engineering Profession for the establishment of public confidence in, and for the technical advancement that will accrue to, the Engineering Profession. State supervision if competently administered assures the public of unbiased and uninfluenced engineering opinion; dispels inherent public fear and apprehension of

the safety of dams; centralizes, and makes uniform, co-ordinated control; and makes available in condensed form recorded technical information and data of inestimable value. State supervision of dams, however, should be ever cognizant of the fact that the advancement of any community depends upon the development of its water resources through the construction of dams. Obviously, this program must not be retarded through overcautious and unwarranted functioning of the office having jurisdiction beyond reasonable requirements for assurance of safety.

H. K. Barrows,[22] M. Am. Soc. C. E. (by letter).[22a]—The writer has read with interest the excellent and full outline by the authors of the various phases of this subject. The list of dam failures is of especial value and very pertinent to this subject.

Recent legislation in Vermont (1929) under which new dam construction has had State supervision is of interest. This Vermont Act provides that no dam impounding more than 500 000 cu. ft. of water in any stream or at the outlet of any body of water shall be constructed without the approval of the Public Service Commission, and requires that application to the Commission for such construction shall include the filing of plans and specifications therefor.

The Commission, with the approval of the Governor, may employ a competent hydraulic engineer "to investigate the property, review the plans and specifications and make such additional investigation as such Commission shall deem necessary."

Under this Act several new projects have been supervised, including the Fifteen-Mile Falls power development, with a dam 170 ft. high. The cost of engineering supervision is paid by the party constructing the dam, through the office of the Commission.

Any existing dam that is unsafe can also be investigated under this Act, and caused to be properly repaired, by the Commission.

In many States the method of supervision through a competent consultant is undoubtedly effective and economical as this class of work is usually not continuous, but nevertheless requires the highest degree of professional judgment and experience. In States where enough work of the kind warrants the continuous employment of an expert it is advisable to have such an engineer under salary by the State.

As regards the extent of inspection during construction, when the work is under the direct supervision of competent engineers for the party building the project, it does not appear necessary or desirable to duplicate inspection. When adequate engineering supervision by the owner is lacking, however, the State inspection should be as complete and adequate as necessary to obtain the desired results.

The writer questions the advisability of incorporating any code relating to plans or type of construction in legislation upon this matter. Legislation should be broad and comprehensive rather than detailed. Such details should be left to the executive authority and its engineer.

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[22] Prof. of Hydr. Eng., Mass. Inst. Tech.; Cons. Engr., Boston, Mass.

[22a] Received by the Secretary April 21, 1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

STEREO-TOPOGRAPHIC MAPPING

Discussion

BY MESSRS. F. H. PETERS, W. H. CROSSON, DWIGHT F. JOHNS, AND DOUGLAS H. NELLES

F. H. PETERS,[14] M. AM. SOC. C. E. (by letter).[14a]—In Canada the ground survey camera has been continuously used for many years in mapping the rugged mountainous areas where numerous commanding positions make this method of survey economically practical for small scale mapping. The method followed is that introduced many years ago and consists of exposing photographs in vertical planes in several directions, usually covering the entire horizon from the particular station. Each photograph serves as a record of the horizontal and vertical angles read from the particular station to the features shown therein. The line joining two stations thus serves as a long base and the map positions and heights of features shown in the area common to the views taken from each of the base terminals are determined by the usual graphical intersection methods. Surveyors mapping in this manner frequently experience difficulty in identifying the same feature or object viewed from different camera stations, and, at times, the intersection angle may be so small as to introduce errors in the graphical intersected positions.

A solution to these objections was offered in the application of the stereoscope for identification purposes and provision for accurate stereoscopic measurement on the photographs exposed in a common vertical plane from the ends of a short measured base. The stereo-comparators devised by Dr. Carl Pulfrich, of Germany, and H. G. Fourcade, of South Africa, were constructed to this end and served for point-by-point fixations. At about the same time the late Dr. E. Deville, Surveyor General of Canada, was experimenting on the construction of a stereoscopic plotter such as might serve for reconnaissance detail mapping from vertical ground views taken in

NOTE.—The paper by C. H. Birdseye, M. Am. Soc. C. E., was presented at the meeting of the Surveying and Mapping Division, Sacramento, Calif., April 24, 1930 and published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1932, by Messrs. Theron M. Ripley, O. S. Reading, and Lowell O. Stewart.

[14] Surveyor-General, Dept. of the Interior, Ottawa, Ont., Canada.

[14a] Received by the Secretary March 2, 1932.

parallel or slightly converging directions. He constructed an experimental model of his plotter ("stereoplanigraph") which owing to certain disadvantages he did not fully develop.

The viewing apparatus consisted of a reflecting stereoscope equipped with a pair of half-silvered mirrors which served as a means of seeing through the eye-holes of the apparatus both photographs of the pair at once, and thus presenting a vertical stereoscopic image of the common area while, at the same time, a "floating mark" or spatial point of light behind the mirrors could be seen directly by the observer. The photographs—diapositives provided with illumination—were adjusted in their holders and with reference to the observing position of the eyes so as to reproduce the internal perspective conditions of the field camera. By further adjusting both the mirrors and the photographs and the inclination of the instrumental eye base the external orientation was reproduced, and a true-to-scale model or plastic image was formed apparently behind the mirrors. The "floating mark" or "cross-wires" consisted of an adjustable black screen, perforated with a hole or slit through which light was transmitted, carried on a movable vertical standard with a pencil at its base. With the "floating mark" at a definite setting for a particular contour, this contour line was plotted by guiding the tracer by hand over the paper so that it remained in apparent contact with the stereoscopic model.

In 1907, Lieut. F. V. Thompson modified the stereo-comparator by adding to it a simple lever system and parallax drum so that, instead of a point-by-point measuring apparatus, it became capable of plotting plan positions automatically, but not contours directly.

The desire to make full use of the camera field of view in covering land areas, to eliminate the waste portions occupied by sky areas prevalent in pairs of ground photographs taken in the same vertical plane, and to obviate the mistakes in point-by-point plotting, resulted later in the design and experimental construction of automatic plotters with facilities for the rapid and economic setting of the photographs in their correct positions for plotting to correspond with the measurements made at exposure, and for the automatic plotting of plans and contours therefrom. In this way the evolution came about of a number of machines among which are found:

In 1908–10, the stereo-autograph by Lieut. von Orell and Carl Zeiss—an instrument still in use and the first to have incorporated in its construction the "Zeiss parallelogram."

In 1919, the autocartograph, (Fig. 9), a large and expensive instrument that served for the precise plotting of inclined or convergent photographs tilted up to 30° below the horizon and exposed on any orientation relative to the base line.

In 1922, the stereo-planigraph (Fig. 6), also a large and expensive machine plotter capable of precise plotting, which allowed for setting photographs exposed on any orientation.

In 1922, the stereotopometer by M. Predhumeau, an instrument of cheaper construction for plotting planimetry and contours from terrestial views exposed in a common vertical plane.

In 1924 the autograph, (Fig. 7), which embodied complete stereo-photogrammetric plotting machinery and was of considerably smaller size and weight than the other plotters of its class. As reconstructed in 1928, it represents one of the best machine plotters at present in practical use with terrestial and aerial photographs.

In 1925, the "stereotopographe," by M. Poivilliers, a plotting machine of the intersection photogoniometric type modeled after the stereo-autograph and autocartograph and recently improved.

The introduction of aerial photography with its problems of unknown exposure conditions—position, tilt, and orientation—started considerable research, resulting in improvements being made to existing plotters in order to take full advantage of the new field as well as the design and construction of new machines for use with aerial photographs.

The aerocartograph (Fig. 8), which has been so well described by Colonel Birdseye, is a much more portable instrument than the autocartograph, (Fig. 9). It will accommodate aerial views taken at any orientation, is capable of turning out precise plotting, and costs less than its predecessor.

In 1925 an Italian "photocartograph" appeared and is still in extensive use, chiefly for large-scale mapping from aerial photographs where a close net of control is supplied. It is constructed on the camera plastica principle and consists of two projectors, each containing a lens, negative carrier, condenser, and light source, all of which are centered on the optical axis of the projector and are rigidly supported by a rod parallel to the optical axis. The rod can be tilted and can be rotated in the tilted position, both movements being centered on the front node of the lens. In addition, the rod is movable in three dimensions relative to the plotting plane and the photograph is rotatable in its own plane. Each projector lens is of similar design, but of shorter focal length than the field lens, and is correctly set for principal distance and principal point.

The four ground-control points of the common overlap of a pair are set out to scale in the plane of the ground glass by means of control markers, and are adjusted in height. Each projector is then set individually by trial (consuming about 20 min.), so that the images of the control points fall in their correct position. When both negatives are set, the control markers are removed and replaced by a ground glass screen, the matte side of which is nearer the projection lenses. A pointer or index marker actuated by the $x, y,$ plotting controls and connected to a pantograph moves over and in contact with the projection surface, and coincidence plotting is effected by "the flicker" system.

The instrument will plot satisfactorily from vertical aerial photographs for a range of height within the permissible depth of focus of the projecting lens, and this range may be increased by extending the instrument base for various plotting zones to bring the point of coincidence within the depth of focus (involving a re-calculation of the height scale and re-setting the pantograph). Photographic extension of control is possible through a few overlaps.

In 1925 Dr. Archibald Barr and Dr. William Stroud began the construction of a "photogrammetric" plotter and later produced one model that was used on experimental work. It embodied the camera plastica projection principle with the depth-of-focus problem obviated by means of an auxiliary lens system. To speed up the setting of the photographs in the plotter, a "tilt-finder" or the "stereogoniometer" designed by H. G. Fourcade was used in locating the photo-plumb points of the pair from the ground-control points, and glass positives printed from the original negatives were perferably used in the projectors. The setting movements of each projector allowed for the re-establishment of the internal perspective of each photograph in relation to the projector lens, and of the external perspective, by a rotation of each photograph in its own plane, the tilting of the projector as a whole about an axis through the front nodal point of the projector lens, and the rotation of the projector as a whole about the normal from the nodal point of the projector lens to the screen. One projector could be moved independently of the other in the X-direction. Furthermore, one projector could be given an offset movement in the Z-direction equal to the difference in altitude of the two air stations.

A stereoscopic comparator was used for the examinations of the separated screen images projected from each of the two plate-holders, which were spaced apart a distance equal to the length of base to the required scale of plotting plus the length of the comparator base.

In this machine the two projectors are moved bodily in the three co-ordinate x, y, z, movements, two of which (x and y) are also imparted to the drawing-board. The auxiliary lens system served to maintain sharp focus between each projector and the screen. The separation of the two projectors in the base direction was to provide ample room necessary for their movements in plotting directly to small scales. Two floating marks at the same separation which serve as real marks in the plane of the screen are contained in the eye-pieces of a stereoscopic comparator used for the examination of the screen images. This comparator could be moved over the screen, but it always maintained the line joining the floating marks parallel to the base direction.

Attached to the comparator is an arm carrying the drawing pencil, and points were easily plotted by simply following, by a single movement, the course of detail as the observer saw it. The drawing-board is close to the operator where he can watch the progress of his work. The screen may be moved bodily up and down, and clutches are provided to enable the drawing-board to be moved independently of the projectors. Both projectors and board are moved by the x, y, and z, control wheels to throw a fresh patch of detail on the screen. Scales for X, y, and z are provided to enable the relative co-ordinates of any point in observed coincidence with the comparator floating marks to be read directly. The first model of this plotter was made for use with lenses of various focal lengths which made it unduly large, but the construction of the machine was quite simple. It was not, however, well adapted to correspondence setting by the Fourcade system and only one model was built.

One of the most recently constructed instruments for use in plotting aerial photographs is the "stereogoniometer." This instrument was designed by Mr. Fourcade using the principle of "correspondence setting" devised by him about 1925. One model of this instrument has been constructed and used for experimental work in the determination of the unknown exposure conditions of stereoscopic air photographs.[15]

W. H. Crosson,[16] Esq. (by letter).[16a]—The author has closely covered the principles of stereoscopic measurements and the operation of modern instruments for making topographic maps by the utilization of aerial photographs in connection with ground control. The application of stereo-topographic principles and the practicability of using stereoscopic instruments on military mapping projects is being studied by Army engineers.

If an instrument can be developed that is capable of rapid delineation of topography from air photographs, the task of the military engineer in time of war will be made easier.

An excellent description of the principles and operation of the aerocartograph is given in Colonel Birdseye's paper. Its operation or rather the amount of map work that could be turned out on this machine would be materially increased if it were possible to indicate on each photograph its exact angle and direction of tilt so that each plate could be set and oriented mechanically in its proper relative position.

Dwight F. Johns, Esq.[17] (by letter).[17a]—Officers of the United States Army who have been concerned with the question of the provision of adequate military maps, recognize that the principles of stereo-topographic mapping, and practical methods of their use under military field conditions, have a definite marked application to the problems of mapping for military purposes.

Topographic maps are necessary for modern military operations. If they are not available for the theater of operations at the outbreak of hostilities, time and space factors will preclude their being produced by the usual ground survey methods—time factors because war will not wait for the topographer, and space factors because the terrain for which maps are most needed, will be that occupied by the enemy. It is a principle of war that offensive action is the only means by which a decision is gained, and offensive action is not secured by staying on one's own side of the fence. Mapping the terrain accessible to topographers is not enough for the necessary offensive operations. A certain amount of it will be necessary, of course, just as a temporary assumption of the defensive may be necessary to become better prepared for the offensive, but the ground that the military forces need to know most about is that occupied by the enemy. This knowledge in past operations has been limited to such pre-war maps as existed, likely to be out of date with

[15] For complete description, see *Professional Paper No. 7*, British Air Survey Committee.

[16] Capt., Corps of Engrs., U. S. Army, Washington, D. C.

[16a] Received by the Secretary March 26, 1932.

[17] Maj., Corps of Engrs., U. S. Army, Fort Leavenworth, Kans.

[17a] Received by the Secretary March 26, 1932.

respect to much of the information needed, to other pre-war intelligence data, and to what could be gained, in a fragmentary way only, by limited ground reconnaissance methods such as patrols, raids, reconnaissances in force with a considerable portion of the command, and by general attacks. At best, these methods could reach only a shallow depth within the enemy lines, and they were usually costly in human life.

Now, the airplane and its subsidiary, the aerial camera, furnish the means by which the necessary information can be obtained, in the form of photographs, within the enemy lines to a depth limited only by the practical photographing radius of the plane, which is many miles.

The aerial photograph itself and the photographic mosaic, are of very material value in such a situation for planimetric study of limited areas and for limited distribution. They do not permit topographic relief to be observed, however, except through stereoscopic observation of overlapping pairs, which would be entirely impossible of course for general use in military operations. Neither do they lend themselves to ready interpretation and study by the general user. Topographic maps which can be quickly and cheaply reproduced in quantity are necessary, and there must be some practical means by which the map can be quickly produced from the air photograph in order that the tremendous advantage of the latter may be retained.

It would appear, then, that the principles of stereo-topographic mapping and such practical field methods of their application as have been and may be developed, will furnish an answer to this important military problem. Colonel Birdseye's clear explanation of these principles, and of the design and operation of the aerocartograph as one instrument embodying them, is a very valuable contribution to military, as well as to civil, engineering literature on this subject.

DOUGLAS H. NELLES[18] (by letter).[18a]—Photo-topographic mapping from land stations was used for practically the entire 30 000 sq. miles mapped in the vicinity of the South-Eastern Alaska-Canada International Boundary Line; for a large portion of the territory adjacent to the 49th Parallel, International Boundary, in the Rocky Mountains; and for the mapping of the great Canadian National Parks in the Rocky Mountains.[19]

When the territory to be mapped has low relief and mapping points are difficult to identify with the ordinary photo-topographic method, good work can be done by the stereo-photogrammetric method. In this method the base between plotting photographs is only a few hundred feet long and points seen in one photograph, can always be identified in those from the other end of the base.[20]

In Canada, mapping from air photographs has been carried on extensively by the Federal Departments in producing what is called "The National

[18] Ottawa, Ont., Canada.

[18a] Received by the Secretary March 26, 1932.

[19] "Photographic Surveying," by M. P. Bridgland, *Bulletin No. 56*, Topographical Survey of Canada, Ottawa, Ont., Canada.

[20] By Messrs. B. J. Woodruff and D. H. Nelles, in *The Engineer* (London), October 30, November 6, and November 13, 1925.

Series Maps" on scales of 1, 2, 4, 8, and 16 miles to the inch. These maps have a very fine numbering system, by which, from a small index map, the number of a map on any scale above, covering any territory in Canada, can be quickly picked out. This system will bear examination by other countries with reference to its application to their own territory. The Royal Canadian Air Force, Civil Operations Section, takes the aerial photographs for the Civil Government Departments under special instructions for each operation. The Air Force has taken approximately to date (1932), 280 000 sq. miles of oblique photographs and 125 000 sq. miles of vertical photographs. In addition, private aircraft companies have taken between 5 000 and 6 000 sq. miles of obliques, and approximately 285 000 sq. miles of verticals for private companies and Provincial Governments. Special mapping methods have been developed in Canada for using aerial photographs.[21]

An accurate survey of the crest of Niagara River Falls, using both ground photo-topographical stations and a special aerial photographic method that is interesting to a student of the subject, has been described by the Geological Survey of Canada.[22]

Colonel Birdseye's paper shows that European methods and instruments are very complicated, and the instruments costly. This fact limits their use on the continent of America to a great extent. The same trend is noticed in European instruments for land photo-topography. On the other hand, surveys in Canada have developed practical methods and simply constructed instruments, made to stand hard usage of day after day of mountain packing but which, at the same time, are capable of giving the highest degree of accuracy. In aerial mapping, Canadians are developing methods and instruments along the same practical lines.

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[21] "The Use of Aerial Photographs for Mapping," *Bulletin No. 62*, Topographic Survey of Canada, Ottawa, Ont., Canada.

[22] "The Niagara Falls Survey of 1927," by W. H. Boyd, *Memoir 164*, Geological Survey of Canada, Ottawa, Ont., Canada.

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DISCUSSIONS

DETERMINATION OF PRINCIPAL STRESSES IN BUTTRESSES AND GRAVITY DAMS

Discussion

BY MESSRS. W. J. STICH, HAKAN D. BIRKE, DIRK A. DEDEL, FRED A. NOETZLI, AND EUGENE KALMAN.

W. J. STICH,[9] ASSOC. M. AM. SOC. C. E. (by letter).[9a]—An interesting adaptation of stress analysis in gravity dams, as first formulated by the late William Cain,[10] M. Am. Soc. C. E., is given in this paper. In addition, the subject of inclined joints and a means of locating these joints has been discussed. As long as dams of only moderate height were proposed, the ordinary methods of stress determination, either graphical or analytical, satisfied the requirements of design. In general, it was considered that: (*a*) If vertical normal stress at the up-stream and down-stream faces of the dam, with an adequate allowance for uplift, appeared to be free from tension; (*b*) if vertical contraction joints were specified at the proper spacing; (*c*) if the sliding factor was kept within accepted limits; and (*d*) if water-tightness and drainage were given due consideration; then, the dam was assumed to be safely designed, and that the selection of the proper profile was a relatively simple matter. With the advent of the high gravity dam, however, and with the resulting limitations imposed on the profile by the bearing power of foundations and the effectiveness of mass concrete in withstanding vertical or inclined cracking parallel to the axis of the dam, the matter of tensile stress within the dam and the provision of inclined joints became of prime importance.

The author's paper appears to be particularly timely in view of the impending construction of dams of great magnitude, such as the Hoover Dam. Furthermore, the progress being made in the art of dam design as a result of experimental research should re-act as a stimulus to the engineer in his efforts to formulate equally advanced methods of theoretical analysis.

NOTE.—This paper by W. H. Holmes, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[9] Designing Engr., Pasadena Water Dept., Pasadena, Calif.

[9a] Received by the Secretary February 5, 1932.

[10] "Stresses in Masonry Dams," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV, (September, 1909), p. 208.

The writer has been unable to check some of the formulas presented. For instance, in order to obtain Equations (26) and (27), it was found necessary to assume S_u equal to S_d and p_u equal to p_d. This, however, is only true for the particular case of $n = 0$ and $m = \frac{1}{\sqrt{k}}$. The formulas for the distance of either face from the neutral line are apparently reversed. If c_1 and c_2 are stated as the distances, respectively, from the neutral line to the up-stream and down-stream faces, then the equations as presented by the author will be valid.

Referring to Figs. 9, 10, 11, and 12, it would appear that no consideration has been given to the effect of the batter of the up-stream face on the base thickness of all wedges and pyramids involved in the calculation of the various trapezoidal volumes.

In July, 1927, the writer was assigned the problem of analyzing the stresses in the proposed San Gabriel Dam, in Southern California. The dam was of the gravity type, arched in plan, with a radius of 1 400 ft. at the reference line and with the cantilevers assumed to take all the load. A tentative profile had been decided upon, based on a limiting foundation pressure of 40 tons per sq. ft. and absence of tension at either the up-stream or the down-stream face. In order to reduce the maximum foundation stress, a rather unusual reverse curve was selected for the down-stream batter near the base; and this batter, together with the height of the dam (492.5 ft.), raised the question as to what objectionable stresses, if any, might exist in the interior.

As a means of calculating these internal stresses, the only available method appeared to be that of Professor Cain. However, this method, when made applicable to the case at hand, was found to be extremely laborious and involved the use of coefficients of eighteen significant figures for the calculation of stresses at the lower elevations of the dam.

In order to expedite the calculations, an approximate method, requiring only the same degree of accuracy as the ordinary calculation of vertical normal stresses and involving only a fraction of the time necessary to apply Professor Cain's analysis, was suggested by Fred A. Noetzli, M. Am. Soc. C. E. By means of this method it was possible, in a reasonable length of time, to make a complete stress analysis and calculation of principal stress trajectories for the dam in question. Subsequently, the writer made a check calculation of the results obtained for two critical sections of the dam (Elevation 450 and Elevation 492.5) by applying Professor Cain's analysis; and, as a further verification of the results, Fredrik Vogt, Assoc. M. Am. Soc. C. E., computed the stresses at these two elevations by means of a more exact method. Both checks confirmed the results obtained by the approximate method.

As the author points out, the necessity of vertical or inclined joints parallel to the axis in high gravity dams is now recognized. The provision for joints at 50-ft. centers in the Hoover Dam is a striking example. This

specification has also been adopted for buttress dams, as evidenced in the design of the Coolidge Dam, Big Dalton (multiple-arch) Dam, in Los Angeles County, California, and in the Rodriguez Dam, on the Tijuana River, for the Mexican Government. If joints are placed along the lines of maximum principal stress, then, theoretically, for reservoir full, the plane of the joint will be one of zero shear, and the calculated stresses will remain unchanged. If, in addition, the joint is grouted adequately and liberal keyways are provided, other stages of the reservoir which may produce moderate shear along this plane need not be a source of concern. The grouted joint will transfer the compression, and the keys will resist the shearing stresses developed.

HAKAN D. BIRKE,[11] JUN. AM. SOC. C. E. (by letter).[11a]—The marked trend of recent years toward the use of high buttress dams has made necessary the development of more accurate and comprehensive methods of stress analysis than were used previously for designing this type of structure. These improved methods of design, which are founded on the work[12] of the late William Cain, M. Am. Soc. C. E., involve an accurate determination of the principal stresses and their directions. Any further developments based on the fundamentals established by Professor Cain should be pursued with caution as such procedures in certain cases may result in misleading conclusions or even erroneous and dangerous results. It is obvious that Mr. Holmes has fallen into some of these pitfalls. The methods developed in his paper are not applicable to the design of buttress dams, and they give incorrect results which are not on the side of safety.

The method, referred to by Mr. Holmes, of determining the vertical normal stress at the heel and toe of a dam is entirely obsolete and of recent years all important dams have been analyzed by the method of principal stresses. For example, in designing the Big Dalton Dam, the Coolidge Dam, the Stony Gorge Dam, and the Rodriguez Dam, methods were developed for determining the principal stresses, which were practical in application and which gave theoretically correct results. Furthermore, in three of these structures a system of inclined joints placed along trajectories of principal stress was used. Relating the location of joints to the direction of principal stresses is, therefore, well established in practice and is not new, as suggested in Mr. Holmes' paper.

The methods developed by the author which involve the determination of stress equations on planes 1 ft. apart vertically have long been discarded in practical designing work. In the first place these methods are entirely too cumbersome for structures with simpler outlines, such as the triangular gravity dam. In the second place these methods give results that are incorrect and not on the side of safety for structures having variable sections, such as buttress designs with flaring sections, haunches, pilasters, arches,

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[11] Designer, Ambursen Dam Co., New York, N. Y.

[11a] Received by the Secretary March 15, 1932.

[12] "Stresses in Gravity Dams," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV, (1909), p. 208.

etc. Grave error will be introduced if these variations are not included in the computations and if the horizonal sections are reduced to simple rectangles as required by Mr. Holmes' methods.

In the case of the gravity dam of triangular section the final procedure of analysis will be greatly simplified if the method presented by the author is slightly modified and carried to its logical conclusion. If the equilibrium Equations (9), (14), and (23) of Mr. Holmes' paper were developed on the basis of taking the three horizontal planes of analysis a distance *dy* apart instead of 1 ft., the intensity and directions of the principal stresses may be found directly by relating these stresses to vectorial planes. The equilibrium Equations (9), (14) and (23) given by Mr. Holmes may be expressed for an infinitely small particle in the following manner:

$$\frac{\partial p}{\partial y} + \frac{\partial q}{\partial x} - k = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (87)$$

and,

$$\frac{\partial p'}{\partial x} + \frac{\partial q}{\partial y} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (88)$$

The state of stress in any structure may be completely determined if any of the stress functions of Equation (87) or Equation (88) are known, provided it is practical to express the variations of structural dimensions by definite

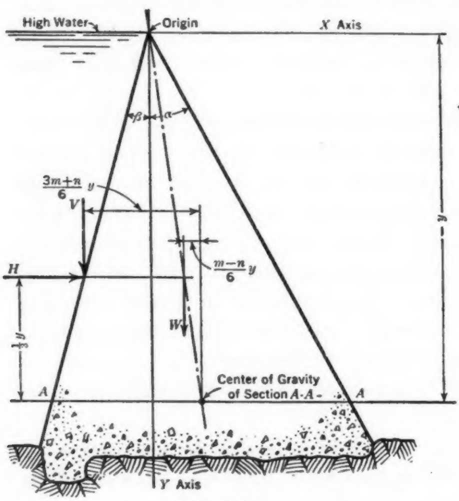

FIG. 14.

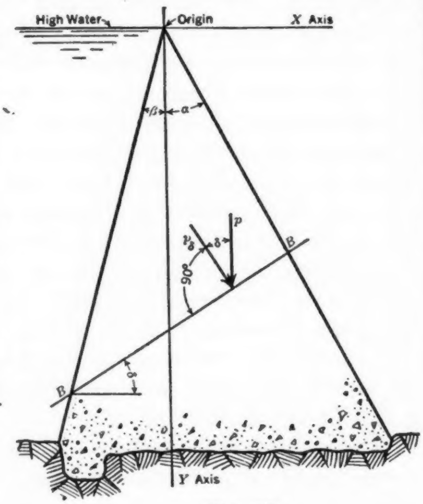

FIG. 15.

mathematical laws. Consider, for example, the triangular gravity dam shown by Fig. 14. The forces acting on any horizontal section, *A-A*, are:

W = weight of masonry $= \frac{1}{2}(m + n)\, k\, y^2$,
V = vertical component of water pressure $= \frac{1}{2}\, n\, y^2$,
H = horizontal component of water pressure $= \frac{1}{2}\, y^2$,

in which, $m = \tan \alpha$ = slope of down-stream face, and $n = \tan \beta$ = slope of up-stream face.

These forces may be combined as follows:

P = sum of vertical forces (masonry + water) on Section A-A, or,

$$P = [(m + n)\ k + n]\ \frac{y^2}{2} \ldots\ldots\ldots\ldots\ldots\ldots (89)$$

Q = sum of horizontal forces on Section A-A, or,

$$Q = \tfrac{1}{2}\, y^2 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (90)$$

M = moment of all forces about the center of gravity of Section A-A, or,

$$M = [2 - (3\,m + n) - (m^2 - n^2)\,k]\ \frac{y^3}{12} \ \ldots\ldots\ldots\ldots (91)$$

These expressions may now be substituted in the well-known stress formula, $p = \dfrac{P}{a} + \dfrac{Mc}{I}$, in which, c = the distance from the center of gravity of the section to the fiber $= x - \dfrac{m - n}{2} y$, and I = moment of inertia $= \dfrac{y^3\,(m + n)^3}{12}$.

Substituting these values in the expression for p and simplifying,

$$p = a_1\,x + b_1\,y \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (92)$$

in which,

$$a_1 = -\ \frac{k\,(m - n)}{(m + n)^2} - \frac{(n^2 + 3mn - 2)}{(m + n)^3} \ \ldots\ldots\ldots\ldots\ldots (93)$$

and,

$$b_1 = \frac{k\,(m^2 + n^2)}{(m + n)^2} - \frac{m - n - 2\,m^2\,n}{(m + n)^3} \ \ldots\ldots\ldots\ldots (94)$$

After determining the normal stress functions from Equations (92), (93), and (94), the shearing stresses may be found by substituting the value of p in Equation (87) and solving for q; thus,

$$q = -\int_x \left(\frac{\partial p}{\partial y} - k\right) dx + \phi_1(y) \ldots\ldots\ldots\ldots\ldots\ldots (95)$$

in which, $\phi_1(y)$ is an arbitrary function of y that may be determined from the boundary conditions at the up-stream or down-stream face. Substituting the value for p in Equation (95), and integrating, $\dfrac{\partial p}{\partial y} = b_1$ and,

$$q = \int_x (b_1 - k)\ dx = (b_1 - k)\ x$$

or,

$$q = (k - b_1)\ x + \phi_1(y)$$

The relation between the stresses at the down-stream face is expressed by $q = m\,p = m\ (a_1\ my + b_1\,y)$ because, for the down-stream face, $x = my$.

Combining these two values for q will give $(k - b_1)\, x + \phi_1(y) = m\, (a_1\, my + b_1\, y)$. Solving for $\phi_1(y)$ gives $\phi_1(y) = [a_1\, m + 2\, b_1 - k]\, my$, and substituting this value for $\phi_1(y)$, in the expression for q:

$$q = a^2\, x + b_2\, y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (96)$$

in which,

$$a_2 = \frac{2\, kmn}{(m+n)^2} + \frac{1}{(m+n)^3}\, (m - n - 2\, m^2\, n) \ldots\ldots\ldots (97)$$

and,

$$b_2 = -\, \frac{kmn\, (m - n)}{(m+n)^2} + \frac{mn\, (m^2 - mn + 2)}{(m+n)^3} \ldots\ldots\ldots (98)$$

The horizontal normal stress may now be determined by solving Equation (88); thus,

$$p' = - \int_x \frac{\partial q}{\partial y}\, dx + \phi_2(y) \quad \ldots\ldots\ldots\ldots\ldots\ldots (99)$$

in which, $\phi_2(y)$ is an arbitrary function of y. Substituting the value for q as found before, integrating, and determining the function, $\phi_2(y)$, will result in the following expression:

$$p' = a_3\, x + b_3\, y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (100)$$

in which,

$$a_3 = \frac{kmn\, (m - n)}{(m+n)^2} - \frac{1}{(m+n)^3}\, mn\, (m^2 - mn + 2) \ldots\ldots (101)$$

and,

$$b_3 = +\, \frac{2\, km^2 n^2}{(m+n)^2} - \frac{1}{(m+n)^3}\, m^2\, (2\, mn^2 - m - 3\, n) \ldots\ldots (102)$$

Equations (92), (96), and (100) are of prime importance as they show that the vertical and horizontal normal stresses (p and p') and the shearing stresses, q, will have straight-line distribution on any plane.

From the foregoing it may now be proved that the stresses normal to any inclined plane making an angle, δ, with the horizontal will be straight and linearly distributed.

The stresses normal to an inclined plane (Fig. 15) will have the value,

$$p\delta = p \cos^2 \delta + p' \sin^2 \delta + q \sin 2\, \delta \quad \ldots\ldots\ldots\ldots\ldots (103)$$

in which, $p\delta$ = stresses normal to the inclined plane, *B-B;* and δ = the angle between the inclined plane and the horizontal.

Combining Equations (92), (96), (100), and (103):

$$p\delta = (a_1\, x + b_1\, y) \cos^2 \delta + (a_3\, x + b_3\, y) \sin^2 \delta + (a_2\, x + b_2\, y) \sin 2\, \delta \quad ..(104)$$

Equation (104) expresses the condition that $p\delta$ varies linearly with the co-ordinates, x and y, along a plane making the angle, δ, with the horizontal. Therefore, the normal stresses along any plane, horizontal or inclined, will be distributed according to the law of the trapezoid. Any section in the dam, that is a plane before the loading, will remain a plane after the deformation, if the deformations due to shearing stresses are neglected.

From the foregoing procedure may be derived an easy method of determining the stress trajectories. The direction of the first principal stress at any point is determined by the equation, $\tan 2\,\theta = \dfrac{2\,q'}{p - p'}$. Substituting the values for q, p, and p',

$$\tan 2\,\theta = \frac{2\,(a_2\,x + b_2\,y)}{(a_1 - a_3)\,x + (b_1 - b_3)\,y} \quad \dots\dots\dots\dots\dots\dots (105)$$

or,

$$[(a_1 - a_3)\tan 2\,\theta - 2\,a_2]\,x + [(b_1 - b_3)\tan 2\,\theta - 2\,b_2] = 0 \dots. (106)$$

If a certain constant value for the angle, θ, is chosen for the inclination of the first principal stress, for example, $\theta = \theta_1$ (see Fig. 16(*a*)), and substituted in Equation (106) the result will be the equation for a curve which

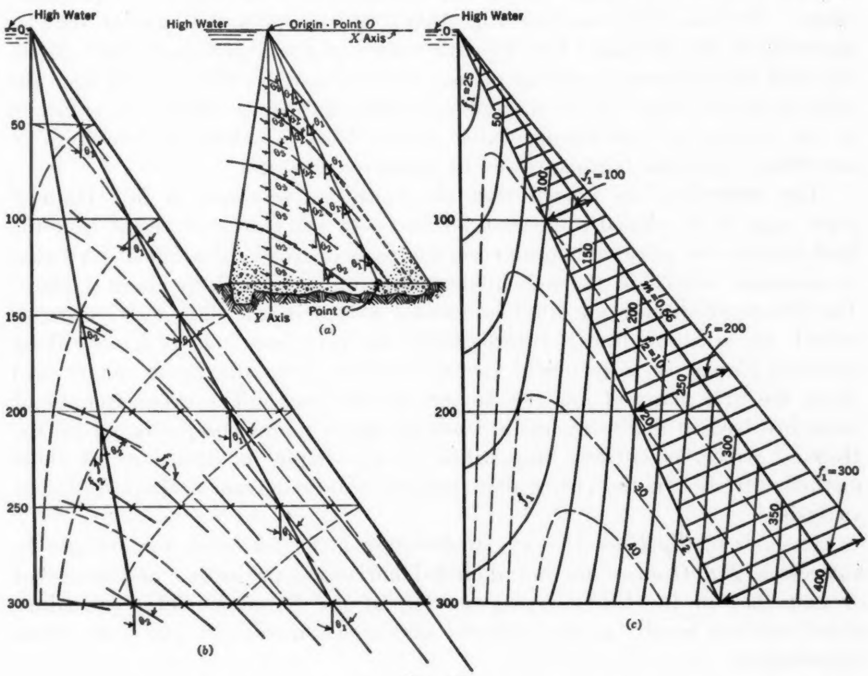

FIG. 16.

the stress trajectories will intersect under a certain constant angle, θ_1, with the vertical. Equation (106) shows that this curve is a straight line through the origin and can easily be located. By determining a few of these straight lines the stress trajectories are fully located. (See Fig. 16(*a*).)

Carrying the analysis further, if the direction of the principal stress is known for one point on a radius vector it will be known for any point on that radius vector. For example, consider Fig. 16(*b*), in which the triangular gravity dam analyzed by Mr. Holmes (Fig. 6) is shown with the stress tra-

jectories and two radius vectors. From this illustration it will be noted that the directions of principal stress are constant along each of these radius vectors.

It will now be proved that principal stress intensities will have straight linear distribution along the vectorial planes as well as the normal stress intensities.

Let the equation for the plane, *O-C* (see Fig. 16(*a*)), be $x = sy$. Substitute this value for x in Equation (104) and let δ be equal to θ_1. This will give,

$$p_{\theta_1} = [(a_1 s + b_1) \cos^2 \theta_1 + (a_3 s + b_3) \sin^2 \theta_1 + (a_2 s + b_2) \sin 2\,\theta_1] y \quad \text{..(107)}$$

Now, p_{θ_1} will be the first principal stress along the plane, *O-C*, and according to Equation (107) it varies linearly along that plane from the value zero at the top. The same thing applies, of course, to the second principal stress. For example, consider Fig. 16(*c*) which shows the gravity dam as analyzed by Mr. Holmes (Fig. 5) with curves of equal principal stress intensity and an arbitrary vectorial plane. It may now readily be seen how the first principal stress along that plane varies uniformly from the value, 0, at the origin to a maximum value at the base, $h = 300$ ft., which is in accordance with the fact expressed by Equation (107).

The foregoing has proved that the methods presented in Mr. Holmes' paper may be developed somewhat farther with the fortunate result that the final application to the triangular gravity dam is greatly simplified. All that is necessary is to find the principal stresses on any single horizontal plane. The principal stresses on other horizontal planes in the dam may be determined by straight linear interpolation of principal stress values along vectorial planes, these principal stresses varying from the values determined along the first plane of analysis to zero at the apex. The great amount of labor involved in analyzing the stresses on many planes, as presented in Mr. Holmes' paper, is entirely unjustified as by simple modifications of these methods stresses in the entire dam may be obtained from a single plane of analysis.

Analysis of Buttress Dams.—In developing the formulas for stresses in buttresses, Mr. Holmes has not included horizontal projection of the arches or haunches in the load-carrying section of the buttress. This procedure gives incorrect results as the haunches and arches materially affect the stress distribution.

For dams of the buttress type only the general equilibrium equations may be set up for elementary prisms, but no expressions for the stresses as functions of the co-ordinates can be developed for the buttress as a whole. The analysis must be made step by step, first determining the shear stresses for a great number of points by taking the sum of the vertical forces on each elementary prism and then, when the shear stresses are known, determining the horizontal normal stresses by summing up all horizontal forces. From the results of the foregoing procedure the principal stresses may be found either graphically by Mohr's circle of stress or analytically by the principal stress equations.

The contraction due to shrinkage and temperature in a gravity dam, or in the buttresses of a buttress type dam, should be taken care of by properly arranged contraction joints. If these joints are placed along the trajectories of the first principal stress and if the second principal stress is zero or some value in compression then there will be no tendency for these joints to open up and as the shear is zero there will be no sliding tendency. Any movement along these joints due to temperature changes or shrinkage will not impair the structure as a whole because the joints still are able to transmit the stresses necessary for the monolithic action of the structure. The writer does not agree, therefore, with Mr. Holmes' statement (see "Distribution of Stresses") that no gravity dam higher than about 100 ft. should be analyzed as a monolith. Since, at all points, the structure is able to take the stresses necessary for monolithic action, why should it not be designed as such? However, as a check, the separate columns into which the structure is divided by the contraction joints may be analyzed independently and then, of course, the second principal stress as found by analyzing the structure as a monolith along the contraction joints should be taken into account as loading.

If the contraction joints (or cracks) have some location other than on a trajectory of principal stress the analysis of the structure will be a difficult problem. For analyzing each monolithic column the forces transmitted through the joints have to be known; but these forces can be determined only by considering the elasticity of the entire structure. When applying Mr. Holmes' equations to the monolithic columns the forces between the columns, that is, the stresses along the joints, must be known. These forces, however, are not known beforehand and, therefor, in the writer's opinion,there is no possibility of using equations of the type set up by Mr. Holmes.

DIRK A. DEDEL,[13] ASSOC. M. AM. SOC. C. E. (by letter).[13a]—It is interesting to note the advances that have been made in dam design, especially in the last decade. The development of the theory of principal stresses applied to dam analysis has shown that the determination of only the vertical normal pressures on horizontal planes does not tell the story of stress distribution.

Mr. Holmes has endeavored to develop a series of formulas for the stress analysis of dams of the gravity and buttress types with the assumption that they could be readily used in solving the principal stresses in these two types. However, in the development of the formulas for the dam of the buttress type, an important factor has been overlooked, making them purely academic and useless in practical design problems. The formula used by the author for vertical normal pressures includes the assumption that the plane on which the stresses are to be determined is rectangular. Of course, this condition never exists in any buttress dam due to the necessity of widening the up-stream end of the buttress to support the flat-slab deck, or to support the arch barrels in the multiple-arch type. It is to be noted also that the round-head buttress type of dam has an enlarged up-stream buttress section to transmit the forces of the water load directly to the buttress proper.

[13] Designing Engr., Ambursen Constr. Co., New York, N. Y.
[13a] Received by the Secretary March 18, 1932.

From Fig. 17 it can be plainly seen that no horizontal buttress section in any of the three types can be assumed as rectangular. The irregular shape of the horizontal buttress section makes it necessary to compute the center of gravity and moment of inertia of each section. With these two factors known and the true eccentricity of the resultant of the vertical loads determined,

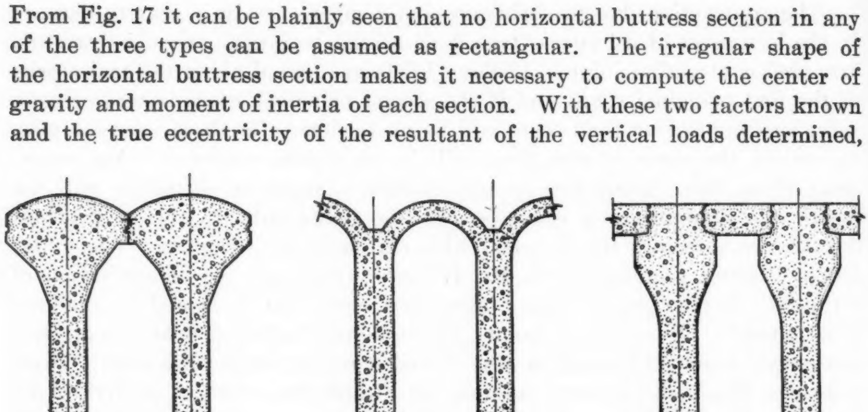

FIG. 17.

the vertical normal stresses at the up-stream and down-stream ends of the section under investigation may be obtained by substituting in the usual formula:

$$\frac{P}{a} \pm \frac{Pec}{I} = p_a \text{ or } p_b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (108)$$

in which, e, the eccentricity, is equal to the distance between the point of application of the vertical load and center of gravity of the section.

The fact that the buttress is irregular in shape makes it impossible to use the author's equations for shear, normal stresses, and principal stresses which include unit buttress thickness and also the factor, b (ratio of buttress thickness). The total area of the section including the haunch area must be used if correct results are to be obtained. To show the large errors in stresses that can be made by the use of an incorrect procedure the writer has analyzed the critical stresses in a massive buttress dam, shown in Fig. 18, using both the correct and incorrect assumptions.

Mr. Holmes' assumptions give entirely too conservative results. Fig. 18 shows an increase of more than 50% in the second principal stress at the maximum height of the dam. It is necessary, therefore, to approach the subject of principal stresses in a somewhat different manner from that presented by the author. The auxiliary stress planes necessary in the computation of these stresses must be a comparatively small distance above and below the plane under investigation, but the use of 1 ft. for this distance

is obsolete. Sufficient accuracy may be obtained if a value of 5 to 30 ft. is used, the exact distance depending entirely upon the height of the structure. However, if computations are made, using the total area of the horizontal section, together with small distances between sections, small differences in large numbers are the result. This makes the work laborious and requires the use of a calculating machine or a many-place logarithmic table to obtain reliable results. Preliminary investigations of stress distinction in a dam do not warrant this vast amount of work. For this reason, a simpler procedure can be used to calculate the critical stresses, which gives sufficient accuracy for most cases.

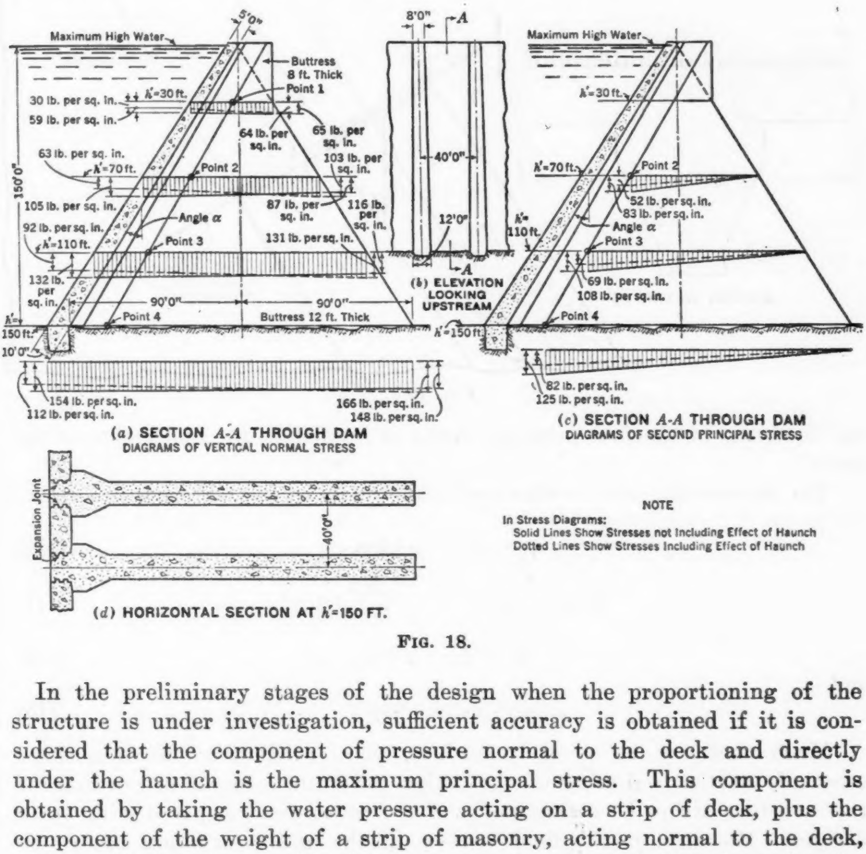

FIG. 18.

In the preliminary stages of the design when the proportioning of the structure is under investigation, sufficient accuracy is obtained if it is considered that the component of pressure normal to the deck and directly under the haunch is the maximum principal stress. This component is obtained by taking the water pressure acting on a strip of deck, plus the component of the weight of a strip of masonry, acting normal to the deck, and considering this combined force to be concentrated directly under the haunch on a strip of buttress, all three elements being 1 ft. wide. If the value of this stress together with the known value of p is combined in Mohr's circle the values of p', q, and f_2 can be obtained, assuming that the direction of the first principal stress is normal to the under side of the deck.

Let N be the buttress spacing; h', the distance below high-water surface; w_w, the weight of a cubic foot of water; W_c, the weight of masonry of a strip,

unity in width, between the points, a and b; α, the angle of the under side of the deck with the vertical; and, t, thickness of buttress; then the first principal stress at Point b, Fig. 19, is,

$$f_1 = \frac{w_w h' N + W_c \sin \alpha}{t} \quad \text{.....................(109)}$$

Knowing the vertical normal pressure and the first principal stress at Point b, the remaining stresses, q, p', and f_2, may be found graphically by constructing Mohr's circle of stresses, keeping in mind the assumption that

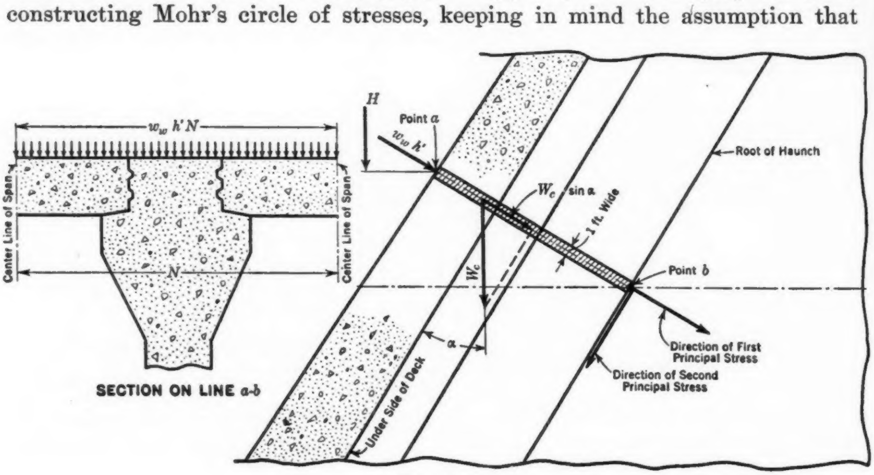

FIG. 19.

the direction of the first principal stress is normal to the under side of the deck.

The stresses may also be expressed analytically by the following equations:

$$q = (f_1 - p) \tan \alpha \quad \text{.....................(110)}$$

$$2\, q_{max.} = (f_1 - p)\,(1 + \tan^2\alpha) \quad \text{.....................(111)}$$

$$f_2 = f_1 - 2\, q_{max.} \quad \text{.....................(112)}$$

and

$$p' = f_1 - q \tan \alpha \quad \text{.....................(113)}$$

Much time can be saved by making these preliminary investigations, especially at the beginning of the design when the general proportions of the structure are to be determined and the economies compared with other designs. However, if the final stresses in the entire structure are to be investigated and the true direction of principal stress trajectories located for the purpose of placing inclined buttress joints, the exact method of analysis similar to the author's method must be used. It is unnecessary to complicate the work shown in Mr. Holmes' paper by introducing various other factors into the stress equations, making them very difficult to use in a practical design problem. Taking into account all the variations in section a few simple equilibrium equations may be constructed, conveniently express-

ing the stress conditions in the buttress. Briefly stated, consider the buttress to be divided into a number of elementary prisms by a series of horizontal and vertical planes. Each prism is then considered in equilibrium by the forces acting upon it and equations may be written expressing the unknown stress in terms of the vertical normal stress, directly determined by Equation (108). If the vertical normal, horizontal normal, and shearing stresses are thus determined, both principal stresses and their direction may be found.

Assume, for example, that it is desired to determine the intensities and directions of the principal stresses on the plane, *B-B*, Fig. 20. An enlarged section of this diagram is shown in Fig. 21. Horizontal planes, *C-C* and *D-D*, are then plotted equal distances above and below the plane, *B-B*, and the

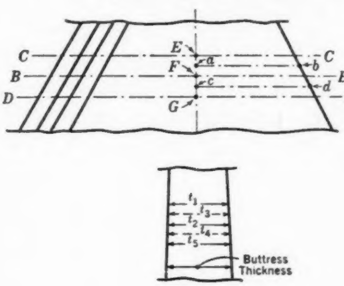

FIG. 20.

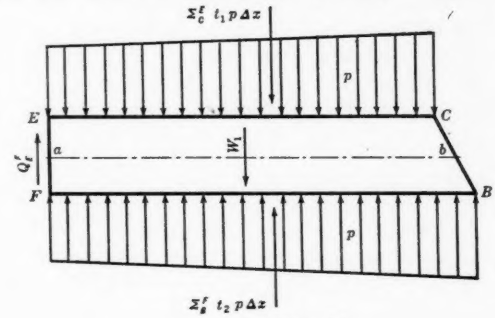

FIG. 21.

values of p are then computed for each plane by Equation (108). Next, construct any vertical plane intersecting the three horizontal planes at Points *E*, *F*, and *G*. The elementary prism, *E-C-B-F*, can now be assumed as separated from the buttress as shown in Fig. 21 and to be held in equilibrium by the external forces acting upon it. These forces are the vertical normal pressure on the plane, *E-C*, $\sum_C^E t_1\, p\, \Delta x$; the weight of the masonry in the prism, w_1, and the total vertical normal pressure on the plane, *F-B*, $\sum_B^F t_2\, p\, \Delta x$. The total shear, Q_{E-F}, across the plane, *E-F*, may now be determined by writing the equation for vertical equilibrium for the prism, *E-C-B-F*, thus,

$$Q_{E-F} = \sum_C^E t_1\, p\, \Delta x + w_1 - \sum_B^F t_2\, p\, \Delta x \qquad \text{.............(114)}$$

The average intensity of shear, q_a, on the section, *E-F*, is equal to Q_{E-F}, divided by the sectional area between Points *E* and *F*. By a similar procedure the average shearing stress, q_c, between Points *F* and *G*, may be determined. The shearing stress at Point *F* on Plane *B-B* is equal to $\frac{q_a + q_c}{2}$.

Next, assume the prism, *a-b-c-d*, to be separated from the buttress and consider it in horizontal equilibrium under the shearing forces acting upon it. As the horizontal and vertical shearing stress intensities are equal, the

total shear acting on the planes, *a-b* and *c-d*, may be determined by the foregoing procedure. The difference between these total shearing forces will equal a normal force, N_y, distributed over an area, *a-c*. These forces are expressed in the following equation:

$$N_y = \sum_a^b t_3\, q\, \Delta x - \sum_c^d t_4\, q\, \Delta x \qquad (115)$$

The average intensity of horizontal normal pressure, p', equals N_y divided by the sectional aera between the planes, *a* and *c*. With the stresses, p, p', and q, determined, principal stresses may be computed by Equation (24) and (25) of the paper.

Although the equations developed by Mr. Holmes for the stress analysis in buttress dams are mathematically correct, the use of incorrect assumptions, together with the introduction of various cumbersome factors, make the formulas of little use for application in practical design problems. Inasmuch as it is possible to make a complete analysis with the use of a few and simple equations this procedure should be followed, because at best the principal stress analysis of buttress dams is laborious.

FRED A. NOETZLI,[14] M. AM. SOC. C. E. (by letter).[14a]—Designers of high dams of the gravity and buttress types have recognized for some time the importance of the determination of the principal stresses in these structures. Mr. Holmes' paper is a valuable contribution to the literature on this subject.

The author's equations are based on the assumption of a linear distribution of the vertical normal stresses upon horizontal planes. Although it is known that the so-called trapezoidal rule is theoretically not quite correct, especially close to the foundation of a dam, this assumption nevertheless may be justified in most cases for the following reasons: First, the difference between the vertical stresses, computed in the ordinary manner by the trapezoidal rule, and the stresses calculated on the basis of non-linear distribution is relatively small, except perhaps for very thick dams; second, shrinkage and temperature stresses in the interior of a dam may amount to several hundred pounds per square inch in tension (at least in some dams these stresses have produced open cracks, so that theoretical refinements regarding distribution of vertical stresses would appear to be of questionable value); and, third, the desirability, if not the necessity, of relieving internal tension stresses by vertical (as in the Hoover Dam), or inclined joints (as in the Coolidge Dam), offers the possibility of an artificial re-adjustment of stresses whereby the validity of the trapezoidal law can be approached relatively closely for the individual units or columns and thereby also for an entire section of a dam. A clear demonstration of a simple method to accomplish this result was described by C. V. Davis, M. Am. Soc. C. E.[15]

Mr. Holmes emphasizes the desirability of providing contraction joints in buttresses and in the vertical slices of gravity dams. The vertical joints

[14] Cons. Hydr. Engr., Los Angeles, Calif.

[14a] Received by the Secretary March 22, 1932.

[15] "Economies in Dams Designed for Increase in Height," *Engineering News-Record*, February 25, 1932, p. 292.

in the Hoover Dam and the inclined joints in the Coolidge Dam are striking examples. These joints are not used merely for the sake of introducing a novelty in dam construction, but are dictated by the necessity of providing for shrinkage in large masses of concrete. Even if no open cracks have developed in a dam, high tension stresses may exist nevertheless. The writer knows of one dam in which vibrations due to blasting operations in the adjoining spillway increased the shrinkage stresses beyond the tensile strength of the concrete, and thereby caused an appreciable extension of cracks which previously had remained stationary for a number of years.

As a remedy against irregular cracks the author recommends contraction joints parallel to the direction of the principal stresses. Such joints divide a dam into inclined columns each of which is supporting its proper proportion of the water load.

The use of inclined columns or struts for supporting the deck of a dam is quite old. The wooden dam shown in Fig. 22 may serve as an illustration.

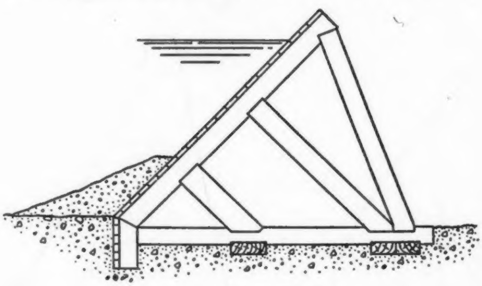

FIG. 22.—TIMBER DAM.

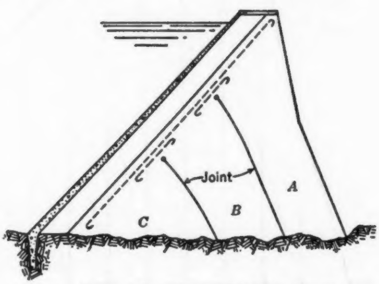

FIG. 23.—BIG DALTON DAM.

Fig. 23 is a typical cross-section of the Big Dalton (multiple-arch) Dam, in California, in which the "inclined columns" are parts of the buttress walls separated by inclined joints.

In the Hoover Dam the joints are vertical (see Fig. 24). The columns are interlocked by key and groove connections. When the concrete has cooled the joints will be grouted in an effort to make the entire dam act again monolithically. A slight modification of joints is shown by the dotted lines in Fig. 24. This arrangement is believed to offer certain advantages over the actual design in that at least the upper portions of the joints extend in planes of little or no shear (practically normal to the down-stream face). In Fig. 24, shear keys are provided in all joints.

Fig. 25 shows another arrangement of joints suitable for dams of lesser height. Rounded openings at the upper end (Fig. 25(*a*)), or at the intersection of the joints (Fig. 25(*b*)), will minimize the danger of an extension of the joints. This expedient has been used with surprising success for stopping cracks in steel castings, etc. In dams, these openings are preferably made to coincide with inspection tunnels.

The author raises the question of how to design a dam which is divided by joints into several columns. In the case of dams illustrated by Fig. 22 the proper method of designing the inclined struts is at once evident. There

is no reason why the inclined "columns" *A*, *B*, and *C*, of the dam shown in Fig. 23 may not be designed in a similar way. If the "open type of construction" is safe, which it undoubtedly is, then the jointed type of buttress illustrated in Fig. 23 should be equally safe, or even more so, by reason of the shear keys and reinforcement tying the columns together. If the monolithic

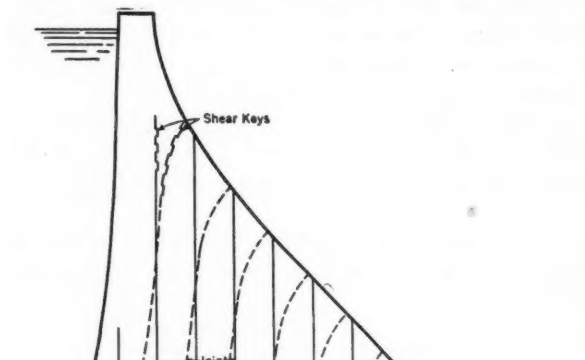

FIG. 24.—HOOVER DAM.

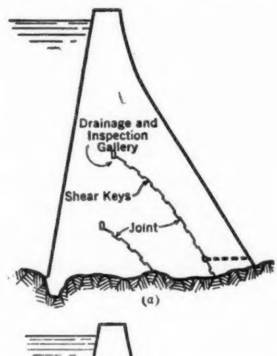

FIG. 25.

character of the structure is more or less re-established by grouting the joints the original assumption of monolithic action under load may be quite close to the actual conditions of the dam.

The designers of the Hoover Dam are satisfied that the vertical joints, properly interlocked and grouted, will not materially affect the size and distribution of the stresses in the dam as computed for the structure without joints. Inclined joints coinciding with planes of zero shear, keyed and grouted for additional safety, should affect still less the size and distribution of the stresses in the dam as a whole. The writer, therefore, is of the opinion that it will be quite proper to design a buttressed or a gravity dam as if it were monolithic, and determine the shears and principal stresses by any of the methods available, for instance, that presented so ably by the author. Joints should then be located at such intervals, say, 50 to 70 ft., as to avoid subsequent shrinkage cracks. The joints are preferably interlocked so that shear forces which may develop in the plane of the joints due to partial or full load are transmitted safely from one column to the other. As a final step of the analysis the stability and safety of the "inclined columns" should be investigated by means of force and string polygons.

EUGENE KALMAN,[16] M. AM. SOC. C. E. (by letter).[16a]—This paper is stimulating from several points of view. Nevertheless, it is doubtful whether the results convey much that is really new or useful to that part of the profession interested in dam engineering.

With commendable industry, the author derives a series of formulas for stresses in the interior of straight and curved gravity dams, and for tapering buttresses of multiple-arch or slab dams; but straight dams of triangular and rectangular profile—as treated in the author's examples—were analyzed successfully more than thirty years ago by M. Lévy,[17] who gave explicit formulas for the stresses at any point of the profile. These formulas, by the way, are more correct than those presented by the author. Furthermore, tapering buttresses were analyzed successfully by Carl Boegh in conformity with the theory of elasticity.[18]

As for the author's formulas for arched gravity dams, the writer thinks that their treatment is a logical contradiction contrary to common sense. Indeed, one may or may not join the group of engineers professing that there is no arch action in an arched gravity dam owing to the fact that vertical radial cracks reduce the dam to a definite number of isolated blocks. If one does join this group, he simply will not design such a dam or derive formulas pertaining to it, and, *vice versa,* if one does not believe in an arched gravity dam cracking up into isolated blocks, he must believe in some arch action which will securely compensate for the tapering of ideal blocks delimited by vertical radial sections. The effect is to eliminate the necessity of taking into consideration the very moderate differences in the behavior of tapering blocks in arched dams and normal retangular blocks in straight gravity dams. If some arch action were not expected, why should a gravity dam be arched, a layout involving increased cost? Conversely, if there is some arch action, why not take advantage of it and omit considering the tedious features of tapering blocks?

The author concludes that his "numerical examples indicate the fallacy of the middle-third theory as a safe criterion for dam design." As a matter of fact, experts on dam design dropped the middle-third criterion almost as far back as the epoch of Rankine; and in recent times no one versed in the subject would ever dream of claiming the middle-third theory gives a safe criterion for dam design.

The writer views with doubt and surprise the industry displayed in this paper and elsewhere with a view to determining stresses in all points of the interior of a dam section. There is legitimate doubt that maximum stresses may be found at points other than on the faces of the dam. Since the publications dealing with the stresses in the interior of gravity dams fill volumes, the writer wishes to show, at least in the case of a triangular profile and with reservoir filled, that there can be no maximum stress in the interior.

[16] Prof., Univ. of Milan, Milan, Italy, now in New York, N. Y.

[16a] Received by the Secretary March 27, 1932.

[17] "Sur l'equilibre élastique d'un barrage en maçonnerie a section triangulaire," also, "Sur la légitimité de la regle dite du trapère dans l'etude de la resistance des barrages en maçonnerie," Comptes rendus de l'Academie des Sciences, Paris, 1898.

[18] "Beitrag zur Berechnung der Spannungsverteilung in Stüttpfeilen von Stauwehren aufgolöster Bauweise," *Beton und Eisen,* Berlin, 1927.

First, consider the following formulas derived by Lévy:

$$\sigma_x = -(\gamma - \gamma' \cot^2 \alpha)\, x + (\gamma - 2\gamma' \cot^2 \alpha)\, y \ldots\ldots (116a)$$

$$\sigma_y = -\gamma' x \ldots\ldots (116b)$$

and,

$$\tau_{xy} = -\gamma' \cot^2 \alpha y \ldots\ldots (116c)$$

σ_x, σ_y, and τ_{xy}, being the vertical and horizontal normal stress and the shearing stress, respectively; x and y, the vertical and horizontal co-ordinates; cotan α, the slope of the down-stream face—the up-stream face being vertical; and γ and γ', the specific weights of masonry and water, respectively. These formulas give the stresses at any point of the triangular profile.

The normal and shearing stresses related to an arbitrary direction having an angle, δ, with the horizontal direction, may be obtained from the stresses, σ_x, σ_y, and τ_{xy}, by the aid of the following well-known formulas:

$$\sigma = \frac{\sigma_x + \sigma_y}{2} + \frac{\sigma_x - \sigma_y}{2} \sin 2\delta + \tau_{xy} \cos 2\delta \ldots\ldots (117a)$$

and,

$$\tau = -\frac{\sigma_x - \sigma_y}{2} \cos 2\delta + \tau_{xy} \sin 2\delta \ldots\ldots (117b)$$

It is known that the principal stresses correspond to that direction for which $\tau = 0$, and that one of these principal stresses will be the maximum stress at the point under consideration.

Suppose, now, that there is a maximum stress at a certain point, P, of the interior, so that its intensity is greater than that of the principal stresses at any other point in that region. Suppose, also, that this particular maximum stress is the principal stress at that particular point, in relation to a direction defined by the angle, θ.

Draw a straight horizontal line through the point, P, and at any point of this line, the vertical and horizontal normal and shearing stresses will be given by Equations (116), in which, x (the distance of the assumed straight line from the top of the dam), will be a constant, while y will be an independent variable. All these stresses, thus, will be linear functions of y only.

Next, consider the intensities of the normal stresses in the direction, θ, at the different points of the assumed horizontal straight line. They will be expressed by Equation (117a), if the angle, δ, is substituted by θ and σ_x, σ_y, and τ are replaced by their values from Equations (116). Since the resulting formula is a linear function of y, it can not yield a maximum value in the interior of the section. If one considers the fact, that at any of the near-by points the stress along the direction, θ, is not a principal stress, while admittedly it is a principal stress at the point, P, under consideration, one sees that the maximum stress at the latter point is surpassed in intensity by other stresses in near-by points, which are not even the principal stresses at those points.

It thus follows that there can be no maximum normal stresses in the interior of the dam; and the same statement holds for the maximum shearing stresses, as can be shown by repeating the foregoing reasoning. If the pro-

file is not triangular, there will be some stress distribution other than linear along straight line sections; but the maximum stress will probably appear at some point on the faces, and probably at some of the lowest points.

In certain cases, structures must be analyzed at every point. A little hole in the center of a sphere subjected to hydraulic pressure causes in that region, a stress three time as great as would be the case if there were no hole. In ordinary beams in reinforced concrete, shearing stress conditions must be studied in the interior to provide for a sufficient number of stirrups, bent-up bars, etc. Dams, however, are supposed to be continuous and homogeneous structures, and, therefore, the writer never believed in searching their interior for extreme stresses.

For the same reason, the generous display of trajectories in modern engineering and especially in dam literature, appears as slightly over-emphasized, when related to their usefulness. Years ago, when he was a young engineer the writer derived much thrill from trajectories of principal stresses; but as yet he has failed to derive from them any practical profit of consequence.

Two statements in the paper should be discussed to some extent, because they are characteristic for a certain peculiar trend of thought among engineers at large. The author claims that a triangular profile with vertical up-stream face is safer than a rectangular profile having the same up-stream face and base. He also states that, in the latter, tension will develop, while there is no tension in the triangular profile.

These statements, of course, are not true. They are the consequence of erroneous assumptions, and there would be little to discuss about them if it were not for the question of why engineers accept similar results. If a farmer, alone, or with the help of a bricklayer, had to build a small dam on his farm, and he had to choose between the triangular and rectangular profile, with identiacl base and up-stream face, he would consider the rectangular profile as safer. Why is it, now, that the engineer accepts the triangular profile as the safer, while the farmer and the bricklayer do not accept it? The reason must be that during his college career the engineer develops a kind of inferiority complex. The bricklayer is a kind of engineer relying on the authority of his common sense. The engineer is a kind of bricklayer, or builder, who, relying on the authority of formulas and mathematical developments and examining professors, relinquishes part of his common sense.

If there is no tension in a triangular profile, and if another triangular mass is superimposed upon the first so as to complete the rectangle, the bricklayer would not believe that such addition would create tension at the base of the dam. On the contrary, he would think that such addition of mass would eliminate or diminish tension if tension existed in the triangular profile; but the engineer believes the incredible.

The author states that in some isolated internal region of a profile tension may develop, while everywhere around that spot the material is compressed. The writer is not able to visualize a mass of homogeneous and continuous concrete of reasonable shape, in which all circumferential portions are compressed, while some central portion is subjected to tension.

Some time ago, Dr. Nicolosi, of the Institute of Technology in Rome, Italy, published a paper showing that in some cases additional haunches between beams and slabs may cause a decrease of strength. Every one familiar with reinforced concrete structures is fully aware of the substantial increase of strength developed by such haunches; and unprejudiced minds would agree that any addition to the section of a beam involves increasing its strength. The bricklayer would agree, and the farmer hewing primitive beams out of logs would agree; but an engineer, or an engineering professor, is often a slave of his formulas, a victim of his acquired inferiority complex.

In a paper[19] discussing Dr. Nicolosi's result, the writer showed that the perfectly correct mathematical operations and deductions were based on a false assumption; that he did not believe in the result even if the assumptions could not be proved to be false; and that the basis of the science of construction would have to be revised if they yielded such results contrary to common sense.

Numerous examples and cases of a similar type could be added. The fact is that many college graduates, in arriving at some paradoxical result, simply limit themselves to a check of the correctness of the mathematical operations, and if the latter are found to be correct, the paradoxical result is admitted as an indisputable fact. Any doubt by outsiders and critics provokes the solemn statement accompanied with an Olympian frown: "You cannot doubt mathematics, can you?"

In the writer's opinion, there is an uneven apportionment in the university curricula between too much mathematics and too little or no effort to apply it to engineering problems, with an emphasis upon the unavoidable presence of physical assumptions and consequent necessity of submitting the final results to the judgment of common sense, or to the verdict of experimental tests.

To return to the author's results under consideration, they are based on the assumption that the vertical normal stress in a horizontal section of a rectangular profile of dam, with reservoir filled, is distributed according to the law of the trapezoid. This is not the case, as shown by Lévy. Consequently, results based on the author's assumption should be faced from the beginning with a certain critical attitude.

The aforementioned result reported by Dr. Nicolosi is based equally on the usual assumption of the validity of the law of the trapezoid. Prompted by a legitimate curiosity as to the validity of that law, the writer undertook some time ago a simple investigation of that problem.[20] He found that the elastic stresses consistent with the postulate of the linear distribution of normal vertical stresses along all horizontal sections of a given profile are algebraic functions of the fourth degree of the co-ordinates, x and y. Hence, the number of such possible elastic conditions is very limited. He showed, particularly, that in no dam profile delimited by straight up-stream and down-

[19] "Su un Paradosso Statico," *Il Cemento Armato*, Milano, May, 1926.

[20] "On Two-Dimensional Linear Elastic States," *Philosophical Magazine*, London, Vol. XII, August, 1931.

stream faces can the stress distribution be linear, except in the triangular profile with the water level at the apex.

In his "Introduction" the author states that "the ordinary method of determining the vertical normal stress at the heel and toe of high dams is not sufficient for the complete analysis of the stresses." This statement is somewhat vague. If Mr. Holmes wishes to convey the idea that the vertical normal stress at the heel and toe (or any other point of the profile), of high, or low, dams, differs from the principal stress at those points, every one will agree with him; but not if he means that it is necessary to review the stresses everywhere in the interior of the profile, as well as at the heel and toe. If Mr. Holmes has at hand some observations to support such a statement with more authority than his numerical result based on the validity of the law of the trapezoidal, such data would be valuable. The author is to be commended for the clear exposition of his results.

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DISCUSSIONS

ECONOMIC PROPORTIONS AND WEIGHTS OF MODERN HIGHWAY CANTILEVER BRIDGES

Discussion

BY MESSRS. HENRY C. TAMMEN, ROBERT W. ABBETT, FRANK W. SKINNER, CHARLES M. SPOFFORD, AND R. WAYNE LINCOLN

HENRY C. TAMMEN,[5] M. AM. SOC. C. E. (by letter).[5a]—The author set out to determine the economic lengths of suspended span, cantilever arms, and anchor arms (or anchor span), for two types of cantilever bridges and in order to lighten his labors has used certain "short-cuts." A check is given for an ordinary cantilever bridge (the author's Type *A*) to show that the short-cuts used do not introduce any consequential error in determining the economic proportions for this type of bridge. The writer is satisfied that this is true for Type *A* cantilever bridges and that, while the increased weights of metal for departures from the most economic lengths of suspended spans shown on Figs. 5 and 7, may not be as large as there indicated, the errors in the economic length of the suspended span can be only trivial. There is doubt as to whether the conclusions regarding Type *C* cantilever bridges are equally as close to the truth and still more doubt whether the curves given in Figs. 12, 13 and 14, correctly show the relative weights of cantilever spans and of simple spans.

In determining the weights of anchor spans for Type *C* bridges the author apparently made a comparison of the average moments in anchor spans of various lengths with the average moment in the 336-ft. suspended span in the Elizabeth Bridge, and assumed that the weights per linear foot of the spans would be proportional to these average moments. This would be approximately true only if the average truss depth remained the same, but it results in errors increasing in magnitude as the length of the anchor span and its average depth depart from these dimensions for the Elizabeth suspended span.

NOTE.—The paper by J. A. L. Waddell, M. Am. Soc. C. E., was published in February, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[5] Cons. Engr. (Ash-Howard-Needles & Tammen), Kansas City, Mo.

[5a] Received by the Secretary March 28, 1932.

Inasmuch as Fig. 10 shows such a decided increase in weight of metal for any variation from the most economical length of anchor span there indicated, it seems probable that the errors cited will have little influence in changing this economical length. They may be of considerable consequence, however, in determining economy of type.

To show clearly the economy of type of bridge, the writer has taken from Figs. 12, 13, and 14, the weights of metal for cantilever bridges and for simple spans, reduced them to equivalent silicon weights, and then calculated their ratios for Type *A* and Type *C* cantilever bridges to the weights for simple spans. Table 8 gives these data and Fig. 17, the plotting of the ratios.

An inspection of the curves for the Type *C* cantilever bridge (Fig. 17) shows that its weight is less than that for the Type *A* bridge for all span

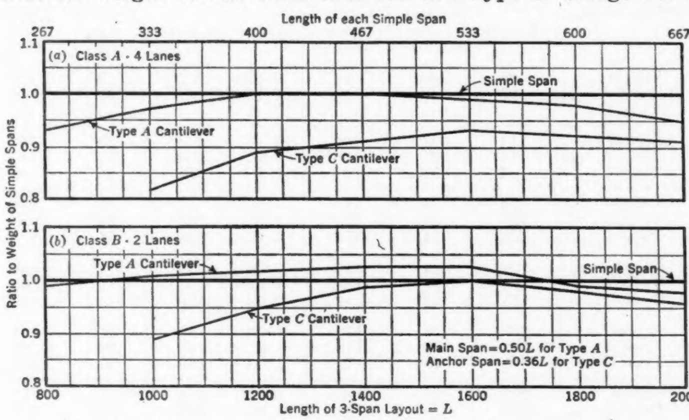

FIG. 17.—COMPARATIVE WEIGHTS OF METAL PER LINEAR FOOT.

lengths. Its weight is also less than a simple span structure, except that for Class *B* two-lane bridges (Fig. 17(*b*)) the curve indicates equality of weight between the Type *C* bridge and a simple span bridge with a span length of about 500 ft. It is surprising to find that the curve has a downward trend to the left for all simple span lengths less than about 500 ft., indicating greater economy by the use of the Type *C* bridge for short span lengths than for long span lengths. The writer would expect the curve to slope upward to the left throughout the range of span lengths considered and he doubts whether the curves for Type *C* shown on these two diagrams represent the true state of affairs. It would appear that the assumptions made in computing the weights to which attention has been called, have introduced errors of such magnitude that the curves shown cannot be considered reliable and that they do not warrant reaching any conclusions as to economy with this type of cantilever bridge.

The same downward trend to the left for spans less than 500 ft. is shown for the curves representing the Type *A* cantilever bridge, but in this case the downward trend is not so sharp. If any conclusion as to the relative economy of simple span and Type *A* bridges is to be drawn from these curves, it is that the two types have substantially equal weights for all spans

up to about 600 ft. Clearly, the author's conclusion that it is economical to change to a cantilever structure at a span length of about 400 ft. is not justified by data presented in this discussion.

It might be emphasized that the comparison being made is for a three-span crossing composed either of three simple spans of equal length or of a cantilever structure in which the anchor arm is equal to one-half the main span. The comparison is, therefore, for a crossing where there is no restric-

TABLE 8.—COMPARATIVE WEIGHTS OF SIMPLE SPAN AND CANTILEVER BRIDGES FROM FIGS. 12, 13, AND 14

Total length of three-span bridge, in feet	Type of span	CLASS B, 2-LANE				CLASS A, 4-LANE			
		Weight	Percentage, carbon	Equivalent silicon*	Ratios	Weight	Percentage, carbon	Equivalent silicon	Ratios
800.....	Simple.....	96	36	91	1.00	113	36	107	1.00
	Type *A*....	95	40	90	0.99	105	40	99	0.93
	Type *C*....	...	..	...	....	...	..	...	....
1 000.....	Simple.....	114	34	109	1.00	131	34	125	1.00
	Type *A*....	116	37	110	1.01	128	37	121	0.97
	Type *C*....	104	38	97	0.89	108	38	102	0.82
1 200.....	Simple.....	134	31	128	1.00	151	31	144	1.00
	Type *A*....	138	34	131	1.02	151	34	144	1.00
	Type *C*....	128	34	122	0.95	134	34	128	0.89
1 400.....	Simple.....	155	28	149	1.00	173	28	166	1.00
	Type *A*....	160	31	153	1.03	174	31	166	1.00
	Type *C*....	153	30	147	0.99	159	30	152	0.91
1 600.....	Simple.....	177	26	170	1.00	198	26	191	1.00
	Type *A*....	182	28	175	1.03	197	28	189	0.99
	Type *C*....	177	26	170	1.00	185	26	178	0.93
1 800.....	Simple.....	205	23	198	1.00	226	23	219	1.00
	Type *A*....	203	25	196	0.99	222	25	214	0.98
	Type *C*....	201	22	195	0.98	210	22	203	0.92
2 000.....	Simple.....	235	20	228	1.00	259	20	252	1.00
	Type *A*....	230	22	223	0.98	248	22	240	0.95
	Type *C*....	224	18	218	0.96	236	18	230	0.91

* Approximate weight of silicon metal that would give same cost as combined carbon and silicon metal in the bridge.

tion on the location of the piers. If there is any restriction requiring the main piers to be placed farther apart than one-third the distance between the end piers, the economy of the cantilever structure will be increased, and a cantilever structure of Type *A* under such conditions may well prove economical for lengths less than the 600-ft. span indicated herein.

For equal weights of a simple span layout and a cantilever layout, the matter of economy in total cost will, of course, have to take account of the substructure, as the author points out, and also of the unit cost of fabrication and erection of the metal work. Due to duplication, the fabrication cost of the simple span should be less than for the cantilever span, so that for equal weights, in order to secure equality in superstructure costs, it follows that the cantilever structure must show a saving in erection cost.

For the Type *C* bridge the author has reached the conclusion that a central span length equal to 0.36 of the total length of the three-span crossing gives the greatest economy. The weights given in the paper are for such

a layout. This arrangement of spans corresponds to a ratio of 0.89 between the end span and the center span, whereas the most economical Type *A* layout has a ratio of 0.50 between the side span and the center span. Considering the rapid increase in weight for the Type *C* layout as the center span is lengthened (see Fig. 10), it may be expected to show economy over the Type *A* layout only for ratios of side span to center span of 0.75 or more.

Simple Spans of Part Silicon Steel.—The author also gives conclusions as to the minimum lengths of simple spans for which the use of silicon steel becomes advantageous. Such determinations, of course, are dependent upon manufacturing processes and conditions, and within one year or five years the quantitative determination of such values might be quite incorrect. Even for conditions as they have been within the last few years, the author's use of a fixed difference between prices of carbon and silicon steels leads, in the writer's opinion, to erroneous conclusions, fixing the span lengths for which silicon steel should be used somewhat too low.

In arriving at proper unit prices to use for carbon and silicon steels, in what is called a silicon steel bridge, due consideration should be given to the fact that the total weight of metal in the silicon steel bridge is somewhat less than in a bridge composed entirely of carbon steel. In such a silicon steel bridge many of the items included in the fabricating cost and the erection cost will remain unchanged in total amount even if the total weight of metal is decreased. To make allowance for this, it is necessary either to apply an increased cost per pound to the carbon steel and to the silicon steel in the silicon steel bridge, or to use the same unit price for the carbon steel in the silicon steel bridge and raise the silicon steel price correspondingly higher. The writer has had occasion to study this problem and submits the following analysis.

The economy of silicon steel for any structure as compared with carbon steel will depend upon the following considerations:

(1) The relation between the allowable unit stresses for silicon steel and carbon steel;
(2) The effect of the lighter weight of silicon steel members in decreasing the dead load stresses;
(3) The effect of the lighter dead load on the substructure, and in the case of movable spans, also on the counterweights, machinery, and other special parts;
(4) The relation between the base costs of silicon and carbon steels; and
(5) The relation between the fabrication and erection costs of silicon and carbon steels.

Of these five items, the third can be ignored for this discussion as the author is considering only superstructure costs. The first and second items also need no consideration herein as they affect only the weights of metal which are already given in Figs. 14 and 15. Some comments, however, will be given later on the second item. This leaves only the fourth and fifth items for consideration.

Base cost of metal here refers to the average cost per pound of plates, shapes, and beams for the bridge, delivered at the site. This cost would

include the cost delivered at the shop and the freight to the site. The difference between base costs of silicon and carbon steel has been given by the manufacturers repeatedly during recent years as $15.00 per ton, or 0.75 cent per lb. At present (1932), this difference may be smaller, but for this discussion it can be assumed as given.

If C_1 is the base cost per pound of carbon steel, in cents; and S_1, the base cost per pound of silicon steel, in cents; then,

$$S_1 = C_1 + 0.75 \qquad (8)$$

The difference between the cost of a pound of metal in place on the structure and the base cost of metal as defined herein will represent the cost of fabrication, the cost of erection, insurance, bond, and contractor's profit. For convenience, this difference will be referred to as the "production" cost. Comparing the relative "production" costs on alternate members of silicon steel and of carbon steel, it will be found that a part of this cost is constant for a member (giving, therefore, an increased cost per pound as a change is made from a carbon member to a lighter silicon member), while the remainder of the cost is variable for the member and will be the same per pound for the carbon member as for the lighter silicon member.

Considering the fabrication cost of two alternate members, the first designed in silicon steel and the second in carbon steel, the same number of rivets would be required for connections, for tie-plates, and for lacing, and, generally, also for riveting together the component parts of the member, such as riveting between angles and plates and stitch-riveting between plates. On heavy members where two plates would be required in carbon steel, frequently one plate will serve in silicon steel, giving a reduction in stitch-riveting; and also where more than two plates are used in carbon steel one less plate would frequently be used in silicon, giving a reduction only in punching. In erection, the field riveting would be practically identical for either member. The lighter silicon members might permit somewhat lighter equipment and perhaps somewhat lighter falsework. The cost of handling completed members either at the shop or in the field will be substantially the same per member. The cost of handling individual plates and angles in the shop prior to assembly in the member might show a saving in total dollars in favor of silicon steel because of somewhat fewer pieces, but this would be offset by the additional attention and inspection necessary to avoid interchange of silicon and carbon parts. The total painting cost, shop and field combined, should be substantially the same for either silicon or carbon steel members. The labor, insurance, bond, and profit can all be taken proportional to labor and material, some parts of which are constant for the job regardless of whether silicon or carbon metal is used and some parts of which will vary with the weight of the metal.

It is difficult to say what part of the "production" cost can be taken as equal per pound of metal for both carbon and silicon steels, but the writer is of the opinion that for span lengths at which it just becomes economical to change to silicon steel (which span lengths would not involve members of great weight), a reasonable value would be 25 per cent. For larger and

heavier structures it no doubt would be increased due largely to the substantial reduction in the number of individual plates and shapes making up the silicon steel members. For the present discussion the value of 25% will be used, but consideration will be given to the possibility that 50% might be more appropriate.

Let E = production cost of carbon steel per pound;
E_1 = production cost of silicon steel per pound;
C = total cost of carbon steel per pound;
S = total cost of silicon steel per pound; and
R = number of pounds of carbon steel required to replace 1 lb. of silicon steel.

Then,

$$C = C_1 + E \qquad (9)$$

and,

$$S = S_1 + E_1 = C_1 + 0.75 + E_1 \qquad (10)$$

but, $E_1 = 0.25\ E + 0.75\ E\ R$; therefore,

$$S = C_1 + 0.75 + 0.25\ E + 0.75\ E\ R \qquad (11)$$

Applying this to the example given by the author for a 420-ft. span, Class B, 2-lane:

Metal in carbon-steel span.................. 164 lb. per sq. ft.
Metal in silicon-steel span:
Silicon .... 95 lb.
Carbon .... 45 lb.

Then, $R = \dfrac{164 - 45}{95} = 1.263.$

Assume that the carbon-steel price of 6 cents is proper and that it is made up as follows: $C_1 = 2.25$, and $E = 3.75$. Then,

$S = 2.25 + 0.75 + 0.25 \times 3.75 + 0.75 \times 3.75 \times 1.263 = 7.49$

The relative costs of the span are then:

Carbon-steel span:
164 lb. @ $0.06.................................. $9.84
Silicon-steel span:
Carbon, 45 lb. @ $0.06.......................... $2.70
Silicon, 95 lb. @ $0.0749........................ 7.12
Total $9.82

This shows the costs to be substantially equal. If 50% of the "production" cost is taken as constant per pound of metal, then,

$$S = 2.25 + 0.75 + 0.50 \times 3.75 + 0.50 \times 3.75 \times 1.263 = 7.25$$

and the cost of the silicon-steel span would be:

Carbon, 45 lb. @ $0.06.......................... $2.70
Silicon, 95 lb. @ $0.0725........................ 6.89
Total $9.59

This shows a small economy by use of silicon steel, and that for Class B, 2-lane bridges, it would be economical to change to silicon steel when the span length is about 400 ft. This compares with the 300-ft. length given by the author. For Class A, 4-lane bridges, similar calculations show that it would be economical to change to silicon steel when the span length is about 350 ft., which compares with a length of 200 ft. given by the author.

The author does not give the general design specifications that were used, nor the ratios that were used between allowable carbon and silicon unit stresses. Presumably, carbon-steel unit stresses are on a 16 000-lb. basis and silicon on a 24 000-lb. basis. Using the latest design specifications for highway bridges of the American Association of State Highway Officials, or the 1929 Conference Specification, both of which put dead load stresses on a 24 000-lb. basis, the span lengths at which it would be economical to change to silicon steel would be slightly increased, due to the smaller amount of truss metal required to carry the dead load of the structure.

At best, such determinations cannot be followed blindly. There may well be cases in which other practical considerations will make the plain carbon-steel span more economical than the carbon-silicon span for lengths greater than those determined by such analyses. The delay which may be caused in securing rollings of a variety of shapes and plates for a comparatively limited tonnage of silicon steel, and like elements, may prove determining factors.

The general conclusion may be offered that while such economic studies as guides to conclusions are desirable tools and are often of considerable use to the engineer who devises them and has an understanding of their limitations, they must be applied by others with caution. Formulas cannot be devised to take the place of ripened experience and judgment.

ROBERT W. ABBETT,[6] ASSOC. M. AM. SOC. C. E. (by letter).[6a]—This study together with Mr. Waddell's previous papers on cantilever bridges for railways should practically settle the question of ideal economic proportions for that type of structure.

The author has placed as the limiting factors for the cantilever bridge the simple span or a series of simple spans on the one side and the suspension bridge on the other. Thus, two important bridge types are omitted from consideration, namely, the arch and the continuous truss. The economics of the arch is so vitally affected by local foundation conditions that it does not lend itself to a *carte blanche* comparison with other types. The continuous bridge on the other hand seems to be rapidly living down its unsavory reputation of the Nineteenth Century, and it now demands recognition.

Given ordinary satisfactory foundation conditions, such as any important structure would require, and with no restrictions as to the location of piers, it is a question in the mind of the writer whether the selection of a series of simple spans is ever justifiable from the point of view of the metal involved. Theoretically, the inherent features of a continuous truss should always cause it to show an economy in material over simple spans. If this is true, where

[6] Instr. in Civ. Eng., Sheffield Scientific School, Yale University, New Haven, Conn.

[6a] Received by the Secretary February 23, 1932.

does the continuous type stand with reference to the cantilever? Giving it consideration would certainly alter the point at which the cantilever should be adopted as determined by comparison with simple spans, and, as the relative economy of the continuous type increases with an increase in span length, a thorough study might possibly reveal facts which would place a new light on the economic selection of bridge types.

FRANK W. SKINNER,[7] M. AM. SOC. C. E. (by letter).[7a]—Too many engineers content themselves with designing safe structures of standard type and ordinary dimensions, with little or no regard to the economic influence of variations in relative proportions. Such variations may involve large differences in efficiencies and total costs that can be determined, almost at a glance, from the graphical data presented in this paper. No engineer, or any other authority, has ever investigated the great field of the economics of steel bridges in a manner comparable with Mr. Waddell's researches; and this paper is a highly important application of its broad principles. It is presented so clearly and simply that, in connection with the author's other books and papers, it gives the most up-to-date instructions for the design of important steel bridges in usable shape for any qualified designer. It provides for a great economy of time and labor for him, and an important reduction of cost of construction, thus notably fulfilling one of the true engineer's most sacred obligations, that of efficient, reliable service to the public, to his clients, and to his chosen profession.

For fifty years engineers have differed widely as to the best relative lengths of the suspended span and its supporting cantilever arms, and the demonstration (under "Tabulation of Results to Present Stage") of the fact that the cantilever arms may each be made as nearly as convenient, about one-quarter the length of the distance between their piers, without materially affecting the most economic results, will certainly be a greatly appreciated fact. This feature will also be in accord with other economic elements, as in the development of field construction methods, where the slow, expensive cantilever erection can be reduced by the erection, under suitable conditions, of longer suspended spans assembled on sheltered falsework, and erected as single hoisted units.

A careful consideration of the clear and forceful demonstrations in this paper will assure most complete reading, confidence in the accuracy of the résumé, and the acceptance and beneficial use of the ten important points thereof will be of great practical and theoretical benefit to designers and critics alike.

CHARLES M. SPOFFORD,[8] M. AM. SOC. C. E. (by letter).[8a]—This paper is an interesting and useful contribution to the literature on highway bridges, and

[7] Cons. Engr., New York, N. Y.

[7a] Received by the Secretary March 12, 1932.

[8] Hayward Prof. of Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.; Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

[8a] Received by the Secretary April 1, 1932.

the conclusions reached should prove of distinct value to engineers engaged in making preliminary studies of long-span bridge projects of the cantilever type.

In his Seventh Conclusion, the author states "it is economic to change to cantilever construction from three simple truss spans of equal length, when that length is about 400 ft." Without questioning the validity of this conclusion, the writer wishes to suggest that it is quite probable that a continuous truss bridge under the same conditions, would be found more satisfactory than, and probably equally economical to, a cantilever bridge provided reasonably good foundations are available. One important advantage which the continuous highway bridge has, as compared with either an end-supported or cantilever highway bridge, is the fewer number of expansion joints in the floor. Such joints are expensive and also frequent sources of maintenance troubles in highway bridges; while the interruptions in the roadway surface due to them are objectionable to motorists. Moreover, the deflection of a cantilever truss is greater in magnitude and rapidity than that of a continuous truss and in one unusually short-span cantilever bridge with which the writer is familiar, the vibration at the ends of the anchor arms under the passage of heavy motor trucks and trolley cars is most annoying to pedestrians. In a continuous structure, also, the hinges which are necessary to make a cantilever structure discontinuous are not required, and the lateral bracing may be made continuous throughout.

From the standpoint of appearance, the writer believes that a continuous structure can be given more pleasing lines consistent with an economical profile than the cantilever with its top chord lines interrupted at the suspended span, as in the cantilevers shown in the paper.

The writer has no comparative figures as to the cost of cantilever and continuous highway spans with modern concrete or other solid floors, as the only continuous structure of moderately long spans with which he is familiar—the Lake Champlain Bridge—was made of the continuous type in order to secure a more pleasing outline than any cantilever or simple span design that could be developed, and to eliminate expansion joints and other expensive and troublesome details. He believes that the construction cost was no more and was probably less than would have been the case with a cantilever design and is quite sure that the maintenance charge will be less and the roadway surface more satisfactory for traffic. One feature of the continuous bridge referred to is the practical elimination of stress reversal in the truss members, but the writer cannot state whether such reversal would be more or less than in a cantilever bridge with the same type of floor and the same loading requirements.

As far as stress computations are concerned, no difficulty occurs in the continuous bridge; a little more labor is required, but the cost of this has no appreciable influence upon the cost of the bridge. No concern need be felt from the effect of pier settlement unless the foundations are so bad that it would probably not be wise to construct any type of long-span bridge upon them. Moreover, much of the settlement of the bridge piers occurs during

construction and is due to the dead weight of the piers themselves and the superstructure rather than to the live load. Hence, by establishing the dead end reactions in a three or two-span structure by jacking at the piers before connecting the trusses, the effect of construction settlement upon the dead stresses may be eliminated.

The writer hopes that in his closure Mr. Waddell will give his views on the relative merits of continuous and cantilever highway bridges and state his opinion as to whether the conclusions reached by him on the economical proportions of cantilever bridges would apply also to continuous bridges.

R. WAYNE LINCOLN,[9] ASSOC. M. AM. SOC. C. E. (by letter).[9a]—The writer has always maintained, that designers are prone to apply the data prepared for railroad bridges formerly published by the author, to any and all kinds of bridges, regardless of loading. It is both surprising and gratifying to note that the economic proportions, as shown in the present paper for highway bridges, are not materially different from those formerly set up for railway bridges.

As the author discusses through trusses exclusively, it would be interesting and instructive if he would make a brief statement concerning the application of these results to deck trusses. In the latter type, the depth of the suspended span generally affects the height of the structure, as was the case with the Lake Union Bridge, in Seattle, Wash., on which the suspended span was made exceptionally short in order to reduce the depth to a minimum and thus save height throughout the entire length of the structure, as the required channel clearance occurred at this span.

Mr. Waddell deserves thanks for the presentation of a paper so timely and requiring such a great amount of preparatory work.

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[9] Assoc. Hydro-Elec. Engr., U. S. Engr. Office, Seattle, Wash.

[9a] Received by the Secretary April 6, 1932.

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DISCUSSIONS

DESIGN CHARACTERISTICS OF THE READING OVERBUILD TRANSMISSION LINE

Discussion

By Messrs. E. E. R. Tratman, and Percival S. Baker

E. E. R. Tratman,[5] Assoc. M. Am. Soc. C. E. (by letter).[5a]—The interesting construction of a transmission-line structure on and over a railway right of way, as described in this paper, inevitably calls to mind the occasional or perennial suggestions for the double-decking of railway lines in city and outlying districts. Some of these projects have been for the purpose of providing additional trackage, as in separating suburban and main-line traffic. More generally they aim at bringing some other railroad (suburban or interurban) into the terminal station in order to improve the service of the secondary line. In at least one case, it has been proposed to build a toll-road viaduct for high-speed automobile traffic above the railroad.

As most of these projects have been put forward by outside interests and would be more or less competitive with the railroad company whose property is to be utilized, it is natural that the railways have not been disposed to regard them favorably. In fact, as a rule such projects have not advanced much beyond the paper or publicity stage. So far as the writer now recalls, none has reached the construction or even the design stage. Nevertheless, the idea or principle has value, and it is quite conceivable that a railway with growing traffic on narrow right of way through a built-up district or through high-priced property might find it economical to double-deck its line, or to co-operate with some other concern in doing this.

One objection that is generally made to such propositions is the danger of damage to the upper structure by derailments on the main level; but accidents of this kind are not frequent enough to constitute a valid objection. Apparently, this objection is given little weight in regard to the Philadelphia power line, since the photographs reproduced in the paper do not indicate that the railroad considered it necessary even to provide guard-rails along the tracks spanned by the tower structure.

As a rule, strong objections are made by railroad companies to structures spanning or crossing their tracks. This applies particularly to crossings by

Note.—The paper by Frederick W. Deck, Assoc. M. Am. Soc. C. E., was published in February, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[5] Western Editor, *Engineering News-Record*, Chicago, Ill.

[5a] Received by the Secretary March 7, 1932.

high-tension electric cables, for which stringent protective requirements have been formulated. It would be of interest, therefore, to have some particulars of the relations and agreements between the railway company and the electric company as to the occupation of the former's right of way and the protection of its traffic. Undoubtedly, the co-operation between the two companies was made unusually easy in this case by the fact that the transmission line will serve the electrification of the Philadelphia suburban lines of the Reading Company.

PERCIVAL S. BAKER,[6] M. AM. SOC. C. E. (by letter).[6a]—The author deals principally with the overbuild on the Norristown Branch of the Reading Company, where the greatest difficulties were encountered. Within the section concerned, there were formerly numerous grade crossings and the Reading Company, at the time the overbuild of the Philadelphia Electric Company was first considered, was engaged in the preparation of detail plans for the elimination of all these crossings through the section of Philadelphia known as Manayunk. The elimination of these crossings involved the raising of the tracks for a total distance of 7 200 ft. For part of this distance, the tracks were carried on an embankment supported by retaining walls and for part on a steel viaduct.

Detail plans of the viaduct and retaining walls were furnished to the Philadelphia Electric Company, which Company then located its towers to conform to the general design.

It was found that the additional tower loads to be supported on the viaduct section were small in comparison with the dead load of the structure and the moving loads of the trains, so that comparatively little additional metal was required in the viaduct and at no place was it necessary to change the type of members as originally designed, although, in some cases, it was necessary to change the details. This is shown particularly in the author's Fig. 6.

Throughout the deck girder part of the viaduct the curb columns were stopped at the bottoms of the transverse girders which were seated on them. Where the overbuild towers occurred, the curb columns were lengthened to the tops of the transverse girders to form direct bearings for the overbuild structures and the transverse girders were framed into the sides of the columns.

On the part of the line where the elevated tracks are carried on earth embankment supported between retaining walls, the tower loads were carried down to the bottom of the walls by means of structural steel foundations, which were embedded in the walls. These steel foundation frames were furnished by the Electric Company and then encased in the concrete wall by the railroad contractor. A careful accounting was kept of all additional costs caused by the overbuild, and this addition was paid by the Philadelphia Electric Company.

The method of erecting the overbuild structures as described by the author proved entirely satisfactory to the Railroad Company.

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[6] Engr. of Bridges, Reading Co., Philadelphia, Pa.

[6a] Received by the Secretary March 12, 1932.

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DISCUSSIONS

STRESSES IN REINFORCED CONCRETE DUE TO VOLUME CHANGES

Discussion

By Messrs. Searcy B. Slack, Charles S. Whitney, and M. Hirschthal

Searcy B. Slack,[7] M. Am. Soc. C. E. (by letter).[7a]—The author has handled very clearly a mathematical analysis of the stresses that should occur in a continuous reinforced concrete structure due to variations in moisture, temperature, and shrinkage. The mathematical analysis, of course, begins with a series of assumptions, and the data on which these assumptions are based are not clearly set forth in the paper.

The assumption is made that the coefficient of expansion of concrete and the coefficient of expansion of steel due to temperature changes are the same. The same factor is used in reducing the formulas. Measurements made on these coefficients show that there is quite a wide variation in the thermal coefficient of concrete. This factor seems to vary between 0.000004 and 0.000006 per degree Fahrenheit. Most of the data indicate that 0.0000050 for ordinary structural concrete is approximately correct. The thermal coefficient of steel has generally been determined to be between 0.0000063 and 0.0000068, the value generally used being 0.0000065. It will thus be seen that there is quite a difference in the thermal coefficients of concrete and steel, and that this difference in the rate of expansion of the two materials is sufficient to cause a rather wide variation in the stresses, that may be expected at different temperatures.

Another factor that should be taken into account in analyzing stresses of this kind is the time at which the analysis is made. If the structure is fairly large, the heat caused by the setting of the concrete will not be dissipated for some time after the concrete has been placed. The initial heat in the concrete causes a rise in temperature in both the concrete and the steel. This occurs while the concrete is taking its final set and bond is developing

Note.—The paper by C. P. Vetter, Assoc. M. Am. Soc. C. E., was published in February, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[7] Bridge Engr., State Highway Board, Atlanta, Ga.

[7a] Received by the Secretary March 5, 1932.

between the concrete and the steel. When this heat due to chemical action of the cement is dissipated and the temperature falls, initial tension is caused in the steel reinforcement due to the fall in temperature. To illustrate: A usual case would be concrete with a thermal coefficient of 0.000005 per degree Fahrenheit and steel with a thermal coefficient of 0.0000065. The difference between the two thermal coefficients is 0.0000015. In a column 2 ft. square, the usual temperature rise would be from 70° to 120° Fahr. This would give a 50° fall in temperature following the set of the concrete; the reinforcement steel would be in initial tension amounting to 2 250 lb. per sq. in. This initial tension would gradually be relieved by the shrinkage of the concrete or the plastic flow, but several months would be required to relieve it. Temperature and stress curves for a case of this kind were presented[8] by the writer in 1929.

The author has presented a very good picture of the bond stress in steel. This picture should be extended or modified to include bond stress developed by the difference in thermal coefficients of the two materials.

CHARLES S. WHITNEY,[9] M. AM. SOC. C. E. (by letter).[9a]—While the author has made a commendable effort to analyze the stresses in a continuous reinforced concrete structure due to volume changes, the writer believes that his basic assumptions are such that his formulas are of little practical value. The following should be considered:

1.—The author assumes the structure to be restrained at its ends so that "the total length of the steel bars must remain unchanged." He further assumes that no restraint exists between the ends, the concrete being free to crack at intervals where it will. He states: "The formulas * * * are applicable to such structures as concrete flumes and canal linings, retaining walls, road pavements, floors in warehouses and large buildings, etc. * * *." As a matter of fact no long structure exists, which is not subject to considerable restraint throughout its lengths from its supports. A continuous wall or pavement is restrained to such an extent by friction of the earth that it would not act as if held only at the ends. The floor of a concrete building is not fixed at the ends, but is restrained by its supports at intervals and cannot be put in the same class as a continuous pavement. In the latter case, the end movement which, of course, must exist, would have comparatively little effect, but the two are similar, inasmuch as they both are restrained to an indeterminate degree by their supports, which fact the author has neglected.

2.—The concrete has been considered as elastic with a definite value of the modulus of elasticity (assumed as 3 000 000 lb. per sq. in.), whereas concrete is to a considerable degree plastic and the effective value of the modulus of elasticity varies greatly with the time and rate of application of the load. It may be less than 1 000 000 for shrinkage and greater than 4 000 000 for

[8] "The Behavior of a Reinforced Concrete Arch During Construction," *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2279.

[9] Cons. Engr., Milwaukee, Wis.

[9a] Received by the Secretary March 28, 1932.

temperature changes at later ages. Plastic flow greatly modifies the effects of shrinkage and temperature changes.[10]

3.—The coefficient of thermal expansion of concrete varies within a wide range and is not usually the same as that of steel, as the author assumes. The difference is so great that considerable stress is produced in the steel and concrete by a large change in temperature.

The problem is very much more complicated than the author's analysis indicates, so much so that it cannot be solved by the formulas presented. No general solution is possible for the variety of structures mentioned. The behavior of actual structures is probably the best indication and, even from them, general conclusions are made uncertain by differences in the properties of the materials, the special arrangement of the structures, and the climatic conditions.

M. HIRSCHTHAL,[11] M. AM. SOC. C. E. (by letter).[11a]—Great credit is due Mr. Vetter as a pioneer in the mathematical analysis of stresses due to volume and temperature changes in concrete—a field greatly in need of such analysis—and for the clear-cut handling of the mathematics and its application. The results obtained by him, however, are surprising, requiring almost prohibitive (economically) percentages of steel reinforcement to meet the condition of eliminating the major effects of volume and temperature changes occurring from the time the concrete is poured until it has been subjected to a cycle of exposure.

Such large results are undoubtedly due to the fact that the author has disregarded or overlooked some of the complications that enter into the problem. The principal factor that was overlooked is that resulting from the condition or fact that these volume changes, either in setting of concrete (or swelling), or in the exposure to rise and fall of temperature, occur not only in the longitudinal direction, L, but at right angles (transversely) in the same plane as well, and, in addition, at right angles to the plane just referred to, resulting in a condition differing materially from that treated by the author.

The shrinkage or temperature stresses at right angles to the direction, L, cause a reduction of the lateral deformation or strain resulting from the shrinkage or contraction due to temperature changes in the direction, L. The relationship between the lateral and longitudinal deformations is expressed by Poisson's ratio. This same ratio would have to be applied when considering the plane at right angles to the surface considered, thus further affecting the deformations respectively caused by the shrinkage stresses in each of the three directions. In other words, a cubical particle in the body of the concrete (a parallelopiped) would be subjected, by the shrinkage stresses, to deformations each respectively reduced by the stresses on the other planes so that the net result would be a considerable reduction of the

[10] "Plain and Reinforced Concrete Arches, a Progress Report on the Limitations of the Theory of Elasticity, and the Effect of Plastic Flow, Shrinkage, Temperature Variations and the Freyssinet Method of Adjustment", Report of Committee 312, Am. Concrete Inst., by Charles S. Whitney, Author-Chairman, *Journal*, Am. Concrete Inst., March, 1932.

[11] Concrete Engr., D. L. & W. R. R., Hoboken, N. J.

[11a] Received by the Secretary March 31, 1932.

value expressed by the symbol z, in the author's analysis. In fact, the value of z would be found, in all likelihood, to lie between one-half and one-third the result derived and yield a percentage of reinforcement more nearly conforming to good practice for that purpose—about 0.25 per cent.

Another factor entering into this analysis is the restraint exerted upon the concrete body in question, preventing its freedom to move under stresses due to shrinkage or temperature changes. This restraint may take the form of vertical support (columns or piers) of horizontal members, such as decks, slabs or parapets, or of counterforts for retaining walls (in both cases these restraining factors may vary in their spacing or distance apart), or it may be the friction of the filling material behind non-counterforted walls, when movement occurs, which also restrains the free movement of the wall to changes due to temperature or to shrinkage in setting.

Still another factor entering into this problem, is that in connection with rise of temperature in concrete in the course of its manufacture due to the chemical interaction of its ingredients. This rise of temperature in concrete, in setting, affects the reinforcing steel so that when the concrete has set and is at the approximate temperature of the outer air, the reinforcing steel will have been subjected to that change of temperature resulting in deformations, including change of length, which will be contrary to the author's assumption that the length of the reinforcing steel in a wall of a certain length remains constant. This will affect the results obtained by the author in Equations (1) to (5).

Following this, the author assumes that "if there are to be no cracks, L must be infinite." This is only true for an infinite length of wall or other structure. For a given wall (without cracks), L is certainly definitely finite; it is the total length of wall, the length between expansion joints (if any), or the length between construction joints which are, in reality, partial expansion joints. A definite value for L will greatly alter the formula derived (Equation (6)) and, of course the subsequent equations derived therefrom.

After deriving the value of p (percentage of reinforcement) in Equation (10), the author remarks that it "is inversely proportional to S_s and z", which appear in the denominator, but z is a function of S'_c, which, in turn, is in the numerator of Equation (10), so that in reality the ratio is not an inverse one, but more or less complicated by this relationship.

Still another factor entering into the problem of stresses due to temperature changes (and for that matter those due to volume changes) is the probable presence of shearing stresses due to the fact that the exposed surfaces are subjected to greater changes of temperature than the laminæ farther removed from the surface, or in the interior of the mass, and the tendency to change their length varies, causing longitudinal shearing stresses which would have a further effect on the existence of pure tensile or compressive stresses assumed.

In assuming values for z, or coefficients of shrinkage, the question of curing must be considered. A value of 0.0005 for z assumes no attempt at curing and, even so, it is somewhat high except for higher strength concretes. Coefficients of shrinkage for concrete that has been protected against the

evaporation of water for a period of ten days have been found to be as low as 0.00012 and with less care in this protection, between the foregoing figure and 0.00025, so that the value assumed by the author is 100% high and, therefore, will result in much higher stresses than would be true for cured concrete. While it is unfortunately true that little attempt is made to cure vertical members, such as columns, piers, walls, or abutments, the fact remains that such curing will result in the elimination of cracks due to that cause, and it is much cheaper to specify some form of curing—as, for example, playing a hose on the surface two or three times a day for 10 to 14 days—than the addition of reinforcement. On horizontal members, such as decks of bridges, there is no difficulty of maintaining curing conditions of one kind or another. As proof, the writer wishes to cite the fact, that on the approaches to the Lackawanna Railroad Bridge over the Hackensack River, consisting of girderless flat slabs on high columns, the contractor was persuaded to use as a curing agent the sand that was later used as the fine aggregate of the next section of viaduct.

Furthermore, the writer desires to call attention to the fact that the coefficient of shrinkage of concrete varies with the cement content as well as with the cement-water ratio. It is not in direct proportion, because the high-strength concretes up to 7 000 and 8 000 lb. per sq. in. (Freyssinet[11] writes seriously of the use of 15 000-lb concrete) behave differently in the matter of shrinkage than the ordinary concretes, and that behavior has not yet been definitely established. It will require careful research work to give more information on the subject of coefficients of shrinkage of higher strength concrete.

This criticism is not intended to detract from the value of the author's work because the writer appreciates the fact that it is one thing to initiate and develop a theoretical analysis and apply it to a practical use, and another to criticize the product thus presented.

While on the subject of shrinkage cracks, the writer wishes to take advantage of this opportunity to refute a statement frequently made, that no parapet or other surface of considerable length can be constructed of reinforced concrete without surface cracks developing at frequent intervals. In connection with improvements along the line of the Lackawanna Railroad since 1922, or before, a number of long flat-slab viaducts have been designed and built with expansion joints 250 to 300 ft. apart, both with fascia columns and with cantilever fascias. In only a few instances have any cracks appeared and these have been traceable to other causes, such as unequal settlement of foundations on compressible soils or on piles. These structures have stood the test of quite a few years of service and without an undue amount of temperature reinforcement.

The paper by Mr. Vetter should tend to emphasize the need of proper curing of concrete so as to avoid the rapid evaporation of the moisture content which is the main cause of the excessive volume changes that occur in uncured concrete.

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[11] *Civil Engineering*, February, 1932, p. 93.

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DISCUSSIONS

WATER PRESSURES ON DAMS DURING EARTHQUAKES

Discussion

BY MESSRS. JOHN H. A. BRAHTZ AND CARL H. HEILBRON, JR., AND BORIS A. BAKHMETEFF

JOHN H. A. BRAHTZ,[24] ESQ., AND CARL H. HEILBRON, JR.,[25] ESQ. (by letter).[25a] —In connection with the design of the proposed Pine Canyon Dam, the writers have been able to follow up lines of thought suggested by Professor Westergaard. The studies made by them are as follows: (1) The author's assumption of an infinitely long reservoir is dropped, and formulas are developed for reservoirs of finite length; (2) the effect of assuming that the water is incompressible is shown; (3) the flexibility of the dam is taken into account and methods of allowing for it in finding earthquake stresses are presented; and (4), an independent method of arriving at the author's principal results is developed.

Effect of Finite Length of Reservoir.—If the reservoir is to be considered as other than infinitely long, several other assumptions may be substituted. Perhaps the simplest is that a distance, L, from the dam there is an immovable vertical wall. Expressed mathematically, this means that the author's Boundary Condition (4) — σ converges toward 0 when x becomes large — is replaced by a new condition, (5) $\xi = 0$, when $x = L$.

A solution of the author's Equations (1), (2), and (5), which satisfies Conditions (1), (2), (3), and (5), is similar in form to the author's solution as expressed in his Equations (15), (16), and (20), differing only in that the factor, e^{-q_n}, occurring in these equations, is replaced by expressions involving hyperbolic functions. Further, the author's Equations (31), (35), (36), (38), and (39) are modified only in that in each equation the factor, $\coth \dfrac{n \pi c_n L}{2h}$, is inserted under the summation sign.

NOTE.—The paper by H. M. Westergaard, M. Am. Soc. C. E., was published in November, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings* as follows: February, 1932, by Messrs. Theodor von Karman and Paul Bauman; and April, 1932, by R. Robinson Rowe, L. Prandtl, and Cecil E. Pearce.

[24] Teaching Fellow, California Inst. of Technology, Pasadena, Calif.

[25] Asst. Engr., Pasadena Water Dept., Pasadena, Calif.

[25a] Received by the Secretary February 3, 1932.

The effect of the use of Boundary Condition (5) is shown in the following tabulation giving the values of the correction factor for Q_0, for various values of the ratio, $\frac{L}{h}$:

Ratio, $\frac{L}{h}$	Correction factor	Ratio, $\frac{L}{h}$	Correction factor
0	∞	1.0	1.095
0.4	1.80	1.5	1.020
0.6	1.37	2.0	1.005
0.8	1.18	∞	1.000

This correction factor is defined as the ratio of the value of Q_0 computed by using the formula based on Condition (5) to the value computed by Equation (31). It will be seen that Q_0 increases not more than 0.5 of 1% if the length of the reservoir is more than twice its depth.

Inasmuch as in any actual earthquake any surface at the far end of a reservoir would probably vibrate with the earthquake, a better assumption than Condition (5) would be that both ends of the reservoir move with the same amplitude and period. To express this mathematically, the author's Boundary Condition (4) is replaced not by Condition (5), but by another condition, (6), $\xi = -\frac{\alpha g T^2}{4\pi^2} \cos \frac{2\pi t}{T}$, when $x = L$.

A solution of the author's differential equations which satisfies Boundary Conditions (1), (2), (3), and (6), differs from the author's solution as before only in the substitution of hyperbolic functions for the factor, e^{-qn}, in Equations (15), (16), and (20). In this case, Equations (31), (35), (36), (38), and (39) are modified by the insertion under the summation sign of the factor, $\coth \frac{n\pi c_n L}{2h} - \operatorname{csch} \frac{n\pi c_n L}{2h}$.

The effect of using Boundary Condition (6) instead of Condition (4) is shown in Table 3.

TABLE 3.—VALUES OF THE CORRECTION FACTOR FOR Q_0.

Ratio, $\frac{L}{h}$	Correction factor	Ratio, $\frac{L}{h}$	Correction factor
0.5	0.397	2.0	0.921
1.0	0.670	3.0	0.983
1.5	0.835	4.0	0.996
		∞	1.000

It will be seen that if Boundary Condition (6) is considered as correct, the error in assuming the reservoir infinitely long is negligible if the length of the reservoir is more than about three times its depth. Incidentally, if the value of L is small compared with the value of h, Boundary Conditions (1), (2), (3), and (6) represent conditions in an open well in the earth or

in a structure during an earthquake, and the expression for the pressure, p, corresponding to Equation (31), reduces to approximately:

$$p = \frac{1}{2} \alpha w L \quad \text{.......................(85)}$$

which means that the water pressure due to earthquake on one side of a well is equal to the "inertia force" of one-half the weight of water in the well.

Since the earthquake waves which cause the motion of the dam and of the up-stream end of the reservoir travel with a more or less definite velocity, the motions at the two ends of the reservoir in an actual case would be out of phase. However, it is shown in Table 3 that if the length of the reservoir is not small, one end has no appreciable effect on the other and, therefore, the phase difference is unimportant. On the other hand, if the length is small enough to affect the pressures, the phase difference must be so slight as to be negligible. Therefore, Table 3 gives a sufficiently close approximation to the effect of finite length of reservoir for all practical purposes.

Effect of Compressibility of Water.—If the author had considered the water to be incompressible, he would have written, instead of his Equation (5),

$$\frac{\partial \xi}{\partial x} + \frac{\partial \eta}{\partial y} = 0 \quad \text{.........................(86)}$$

Using Equation (86), he would have obtained exactly the same Equations (15), (16), and (20), except that the coefficients, c_n, the value of which is given in his Equation (21), would have the value, unity. Continuing, in his Table 2, he would have only one set of values applying to all heights of dam, and these would be the same as he has listed for "small" heights of dam. In other words, therefore, the compressibility of the water is of no importance except when the dam is high.

Effect of Flexibility of Dam.—The author states:

" * * * since the period of free vibrations of the dam, T_0, is usually a fraction of a second, while the period of vibration, T, of the earthquake may be assumed to be not less than a second, * * * resonance * * * need not be expected ordinarily, although the possibility of part resonance may well be investigated in some cases of high dams."

Approximate formulas for calculating the effects of this part resonance have been developed by the writers. As part resonance is caused both by the hydrodynamic pressures and by the inertia effect of the weight of the dam, and as it adds to both the hydrodynamic pressures and the inertia forces, it is impossible to separate the two effects and both must be considered.

The procedure, in general, is as follows: The hydrodynamic pressures are first taken as those given by the author's formulas and the inertia forces are taken as if the dam were rigid; the deflections of the dam due to these forces are found; the displacement of the dam is now taken as the displacement of its base plus the deflection; the hydrodynamic pressures and the inertia forces are recalculated on the basis of the new displacements; and the process is repeated if a closer approximation is desired.

The most difficult part of the procedure is the solution of the author's differential Equations (1), (2), and (5) for a new set of boundary conditions.

Conditions (1), (2), and (4) are unchanged. Condition (3) expresses that the face of the dam moves in a simple harmonic motion with the amplitude of the motion of the earth's crust. If the dam is flexible, only the base moves with the prescribed motion, and Condition (3) is replaced by some relation of the general form of Condition (7), $\xi_0 = f(y, t)$, in which, ξ_0 is the value of ξ for $x = 0$.

If the function, $f(y, t)$, of Condition (7) is considered as known, then a set of expressions for ξ, η, and σ, can be found which satisfy the differential equations and Boundary Conditions (1), (2), (4), and (7). These expressions need not be written here; it can be shown that they lead to equations for p and ξ_0 of the forms:

$$p = \frac{4\pi^2}{T^2} \frac{w}{g} \sum_{1,3,5..}^{n} E_n \sin \frac{n \pi y}{2h} \quad \ldots\ldots\ldots\ldots\ldots\ldots (87)$$

and,

$$\xi_0 = -\frac{\pi}{2h} \cos \frac{2\pi t}{T} \sum_{1,3,5..}^{n} n c_n E_n \sin \frac{n \pi y}{2h} \quad \ldots\ldots\ldots (88)$$

in which, the factors, E_n, are constants to be determined so as to satisfy Equation (88). If the values of E_n are found, then p can be computed.

In order to find these values in Equation (88) it is desirable to have the function, $\xi_0 = f(y, t)$, in as simple a form as possible; it is impracticable to use an expression for ξ_0 involving several different powers of y. In choosing the form of the function, $f(y, t)$, it is convenient first to consider the very special case of a rectangular dam of height, H, the base of which moves according to the author's Equation (7), and which is flexible in shear but not in moment. It can be shown that the motion of such a dam is expressed by:

$$\xi_0 = -r \frac{\cos \nu \gamma y}{\cos \nu \gamma H} \cos \nu t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (89)$$

in which, $\nu = \frac{2\pi}{T}$, $\gamma = \sqrt{\frac{\rho}{Gg}}$, ρ is the density of concrete, and G is its elastic modulus in shear. It is seen that in this case the motion of every part of the dam is in phase with that of its base; and it can be shown that in the more general case this must also be true, so that the function, $f(y, t)$, is $\cos \nu t$ multiplied by a function of y.

If ξ_0 of Equation (89) is plotted as a function of y for a given time, a curve, as shown in Fig. 15(a), is obtained. For a triangular dam, the shear deflections due to either inertia or hydrodynamic forces form curves of similar shape, while the moment deflections due to both effects form curves of the general shape of Fig. 15(b). The total deflections form curves of the shape shown in Fig. 15(c). This curve is nearly a straight line, and in most cases, little error will be introduced by assuming that it is straight.

Making this assumption, and with δ_0 equal to the maximum deflection at the top due to dynamic forces, the expression for $f(y, t)$ must give $r \cos \nu t$ for $y = H$, and $(r + \delta_0) \cos \nu t$ for $y = 0$; therefore,

$$f(y, t) = \xi_0 = \left[r + \delta_0 \left(\frac{H - y}{H} \right) \right] \cos \nu t \ldots\ldots\ldots\ldots (90)$$

Substituting this value in Equation (88),

$$r + \delta_0 \left(\frac{H - y}{H} \right) = \sum_{1,3,5,\ldots}^{n} E_n \frac{n \pi c_n}{2h} \sin \frac{n \pi y}{2h} \ldots\ldots\ldots\ldots (91)$$

It can be shown that Equation (91) is satisfied, within the limits, $0 < y < h$, where it is applicable, by,

$$E_n = \frac{8h}{n^2 \pi^2 c_n} \left[r + \delta_0 + (-1)^{\frac{n+1}{2}} \frac{2h}{n \pi} \frac{\delta_0}{H} \right] \ldots\ldots\ldots\ldots (92)$$

If $\delta_0 = 0$, the substitution of E_n from Equation (92) in Equation (87) gives exactly the author's Equation (31).

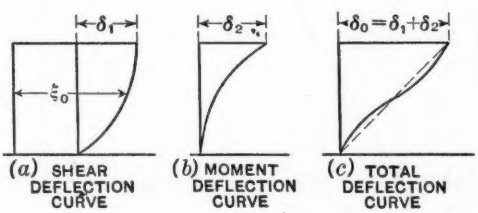


FIG. 15.

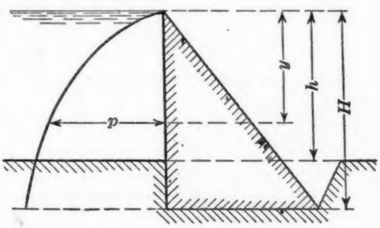


FIG. 16.

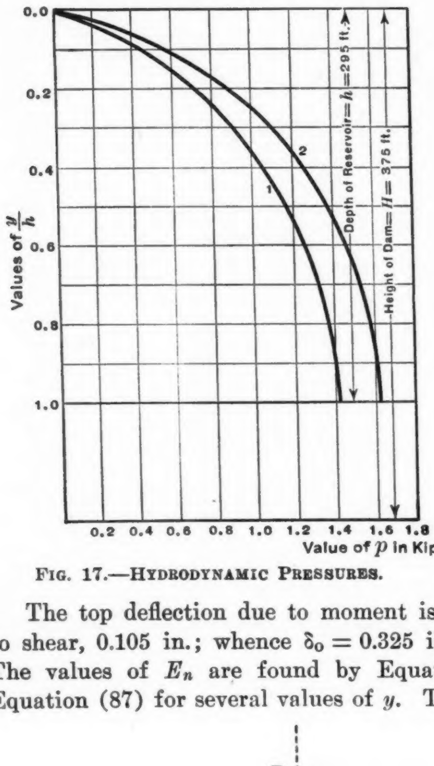
The inertia coefficient, α_1, defined as the maximum acceleration of any part of the dam divided by g, is $\frac{1}{g}\left(\frac{\partial^2 \xi_0}{\partial t^2}\right)$ for $t = 0$. Under the straight-line assumption this becomes,

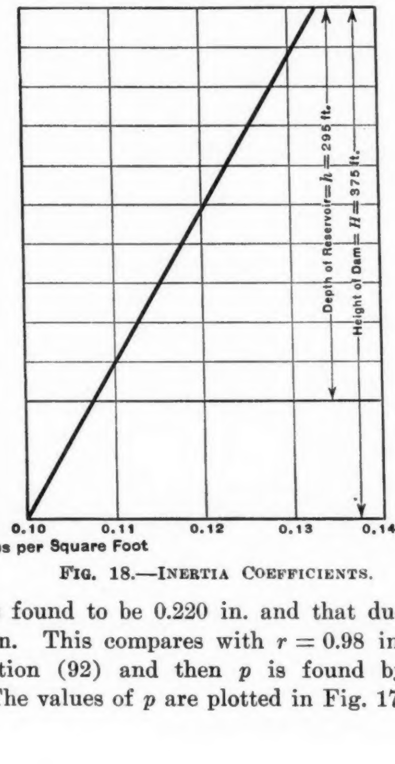
$$\alpha_1 = \alpha \left(1 + \frac{\delta_0}{r} \frac{H - y}{H} \right) \ldots\ldots\ldots\ldots\ldots\ldots (93)$$

It will be noticed that a distinction is made between the depth of the reservoir, h, and the height of the dam, H. If the up-stream face of the dam is back-filled to the original stream bed, the depth, h, is probably best measured down to this point. The height, H, is measured down to the foundation of the dam, or lower if it is desired to allow for foundation deflection (see Fig. 16).

The method and formulas here given have been applied to a straight masonry dam of triangular section, in which, H = 375 ft., h = 295 ft., the base is 0.85 of the height, the concrete weighs 150 lb. per cu. ft., the elastic moduli in bending and shear are 2 000 000 lb. per sq. in., and 900 000 lb. per sq. in., respectively. The reservoir is assumed to be full to the apex of the triangle, and an earthquake is assumed, for which α = 0.1 and T = 1.0 sec.

The deflection, δ_0, at the top of the dam is determined in the first approximation by applying the hydrodynamic forces for an assumed rigid dam in accordance with the author's parabolic distribution and the inertia forces for a rigid dam. The hydrodynamic forces are computed for $h = 295$, and the parabolic distribution is assumed to extend also over the lower part of the dam, where $h < y < H$.





Value of p in Kips per Square Foot

FIG. 17.—HYDRODYNAMIC PRESSURES. FIG. 18.—INERTIA COEFFICIENTS.

The top deflection due to moment is found to be 0.220 in. and that due to shear, 0.105 in.; whence $\delta_0 = 0.325$ in. This compares with $r = 0.98$ in. The values of E_n are found by Equation (92) and then p is found by Equation (87) for several values of y. The values of p are plotted in Fig. 17,

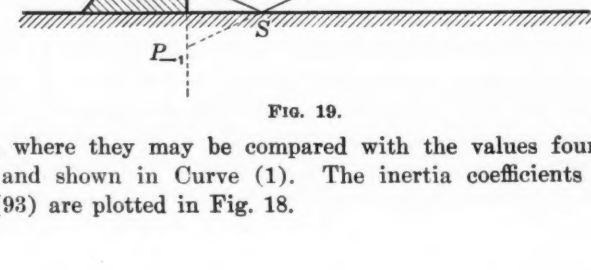


FIG. 19.

Curve (2), where they may be compared with the values found by Equation (31) and shown in Curve (1). The inertia coefficients as given by Equation (93) are plotted in Fig. 18.

In the case of a dam of this height the increase in pressure due to flexibility is small and it appears unnecessary to carry the computations through another cycle. The shears, moments, and stresses in the dam due to earthquake are to be computed by applying the hydrodynamic pressures represented by Curve (2), Fig. 17, and the inertia forces corresponding to the inertia coefficients in Fig. 18.

In order to place confidence in the author's result, it seems in keeping to explain the true physical picture, and to show that the use of a simple application of it produces the same result and, therefore, strengthens and proves the original result correct.

A point, P_0 (Fig. 19), in the face of the dam is considered to be emitting water at a rate that varies with time. The effect of this emission at some point, R, is to be found. In the first place, waves of compression travel from P_0 directly to R. Secondly, waves travel from P_0 along $P_0\,Q$ and, on reaching the free surface of the reservoir, Q, are reflected along QR in opposite phase, seeming to come from a point, P_1, at a distance above the sur-

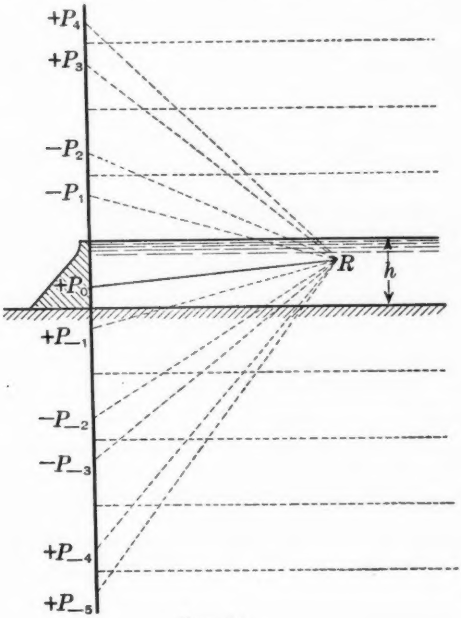


FIG. 20

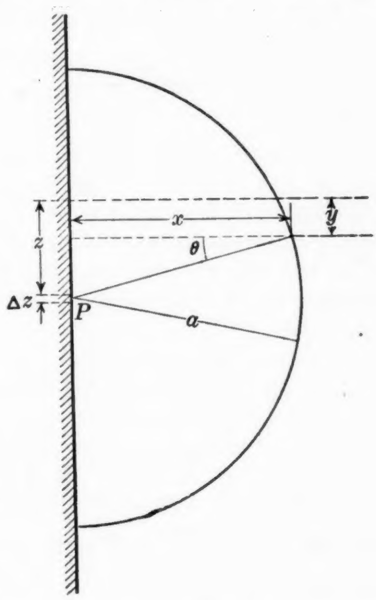


FIG. 21

face equal to that of P_0 below it. Thirdly, waves travel along $P_0\,S$ and are reflected at the bottom of the reservoir along $S\,R$ without change of phase, seeming to come from a point, P_{-1}, at a distance below the foundation equal to that of P_0 above it. Furthermore, waves from P_0 travel in other directions and after being reflected two, three, or more times, reach R. The total effect of the emanation at P_0 upon the point, R, in the reservoir is the same as the sum of the effects of equal emanations at the infinite series of points $-P_{-3}$, P_{-2}, P_{-1}, P_0, P_1, P_2, P_3— shown in Fig. 20 on the face of the dam extended,

acting in a medium unbounded except at the face of the dam extended and in phase or out of phase with P_0, according as they are marked plus or minus in the diagram.

The emanation at any point, P (Fig. 21), is now to be considered as due to the motion of a part of the face of the dam of height, Δz, moving with an acceleration, $\alpha\, g$. Then, the rate of change of emission of water is,

$$\frac{dQ}{dt} = \alpha\, g\, \Delta z \qquad (94)$$

If the water is unbounded except along the one line the effect of this emanation will be the same at all points on a semi-circle of radius, a, and center at P, and if the water is considered incompressible, the radial acceleration, $\frac{dv_r}{dt}$, at a point, R, on the semi-circle is,

$$\frac{dv_r}{dt} = \frac{1}{\pi\, a}\, \frac{dQ}{dt} = \frac{\alpha\, g\, \Delta z}{\pi\, a} \qquad (95)$$

The vertical component of this acceleration is,

$$\frac{\partial^2 \eta}{\partial t^2} = \frac{dv_r}{dt} \sin\theta = \frac{\alpha\, g\, \Delta z}{\pi\, a} \sin\theta = \frac{\alpha\, g\, \Delta z}{\pi\, a}\, \frac{z - y}{a} \qquad (96)$$

and,

$$\frac{\partial^2 \eta}{\partial t^2} = \frac{\alpha\, g\, \Delta z}{\pi}\, \frac{z - y}{x^2 + (z - y)^2} \qquad (97)$$

If, now, R is a point in the reservoir, the total effect of emanation at a point, P_0, is as shown, the sum of the combined effects of all the points, P (Fig. 20), being considered as acting in an infinite medium; and the vertical acceleration at R is,

$$\frac{\partial^2 \eta}{\partial t^2} = \frac{\alpha\, g\, \Delta z}{\pi} \sum_{-\infty}^{+\infty} n\, (-1)^n \left[\frac{(2\,n\,h + z) - y}{x^2 + \left\{(2\,n\,h + z) - y\right\}^2} - \frac{(2\,n\,h - z) - y}{x^2 + \left\{(2\,n\,h - z) - y\right\}^2} \right] \qquad (98)$$

If the whole face of the dam is assumed to move with the acceleration, $\alpha\, g$, the value of $\frac{\partial^2 \eta}{\partial t^2}$ at R is given by Equation (98), with Δz replaced by dz and integrated from $z = 0$ to $z = h$.

Performing the integration and reducing,

$$\frac{\partial^2 \eta}{\partial t^2} = \frac{\alpha g}{\pi} \sum_{-\infty}^{+\infty} n\, (-1)^{n-1} \log_e [x^2 + (2\,n\,h - y)^2] \qquad (99)$$

In order to find the pressure at a point the author's Equation (2) and Condition (1) are necessary; if the former is integrated with the latter as a constant of integration, there results,

$$\sigma = \frac{w}{g} \int_0^{y} \frac{\partial^2 \eta}{\partial t^2}\, dy \qquad (100)$$

Substituting Equation (100) in Equation (99),

$$\sigma = \frac{w\,\alpha}{\pi} \sum_{-\infty}^{+\infty} n\,(-1)^{n-1} \int_0^y \log_e [x^2 + (2\,nh - y)^2]\, dy \ldots\ldots (101)$$

Integrating,

$$\sigma = \frac{w\,\alpha}{\pi} \sum_{-\infty}^{+\infty} n\,(-1)^n \left[2\,nh \log_e \frac{x^2 + (2\,nh - y)^2}{x^2 + (2\,nh)^2} - y \log_e \left\{ x^2 + (nh - y)^2 \right\} \right.$$
$$\left. + 2\,y + 2\,x \left\{ \tan^{-1} \frac{2\,nh - y}{x} - \tan^{-1} \frac{2\,nh}{x} \right\} \right] \ldots\ldots\ldots (102)$$

This is a general expression for σ. In particular, at the face of the dam, $x = 0$, and,

$$-p = \sigma = \frac{w\,\alpha}{\pi} \sum_{-\infty}^{+\infty} n\,(-1)^n \left[2\,nh \log_e \frac{(2\,nh - y)^2}{(2\,nh)^2} - y \log_e (2\,nh - y)^2 + 2\,y \right] . (103)$$

Equations (102) and (103) yield exactly the same numerical results as the author's Equations (20) and (31), respectively, when the C'_n values are taken equal to unity.

The foregoing derivation assumes the water incompressible. To follow the same method with compressible water it would be necessary to use Bessel's functions, which are unfamiliar to most engineers.

BORIS A. BAKHMETEFF,[26] M. AM. SOC. C. E. (by letter).[26a]—The analysis of water pressure on dams during earthquakes presented in this paper should be considered a pioneer achievement in a realm heretofore unexplored. The engineer is often aided in gaining a grasp of new concepts by comparisons in terms of more familiar objects in other fields. For example, in studying the additional water pressure on dams caused by earthquake shocks, comparisons may be made with the well-known phenomenon of water-hammer in conduits caused when the velocity of flow changes rapidly.

In both cases, the problem deals with acoustical waves. For example, Equations (1), (2), and (5), in the paper, which the author integrates for given boundary conditions, are the general acoustical equations that Joukowsky[27] and Allievi[28] used for a general mathematical study of water-hammer. On the other hand, water-hammer lends itself to elementary treatment along the general lines suggested by St. Venant for the study of elastic impulses in bars[29] and of surface waves in open channels.[30]

[26] Prof. of Civ. Engr., Columbia Univ., New York, N. Y.

[26a] Received by the Secretary March 19, 1932.

[27] *Proceedings*, Academie des Sciences (St. Petersburg), Tome IX, 1900; also *Journal*, Am. Water Works Assoc., 1904.

[28] *Annali*, Soc. Ing. ed Arch. Ital., 1901; *Revue de Mecanique*, 1904.

[29] See, for example, Donnell, *Transactions*, Am. Soc. Mech. Engrs., Vol. 52, No. 22, 1930.

[30] Comptes rendus de l'Académie des Sciences, 1870.

Professor Westergaard's "preliminary" case, in which only the horizontal motion of the water layers is considered, is mechanically identical with the model used by Flamant[31] for the study of compression waves in connection with water-hammer. In analyzing the additional pressure, P, created in a conduit by a sudden reduction of the velocity of flow, v, Flamant reverses the movement and determines the pressure, caused by setting in motion a piston sliding in a cylinder that is filled with fluid originally at rest. (See Fig. 22.) By applying the momentum equation in connection with the continuity

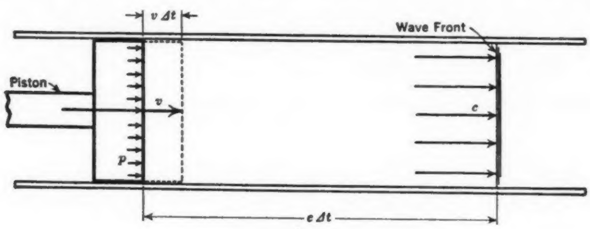


FIG. 22

principle, it is found that the motion of the piston creates a compression wave that is propagated with a speed, c. The pressure exerted on the piston may be stated in terms of the velocity, v, of the piston by the simple equation:

$$\frac{p}{w} = z = \frac{c}{g}\,v \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (104)$$

in which, w is the weight of a cubic foot of the liquid; g is the acceleration due to gravity; and z is the pressure head measured by the height of a column of fluid which corresponds to the water-hammer pressure, p. If the walls of the cylinder are assumed to be absolutely rigid, the movement will then be identical with that in the author's preliminary case. The dam will act as the piston, while the speed, c, will be the acoustical velocity, 4 715 ft. per sec. The numerical value of the ratio, $\frac{c}{g}$, will be $\frac{4\,715}{32.2} = 147$ sec. The significance of this value of the ratio, $\frac{c}{g}$, is that for every foot of velocity lost in a penstock or created by the motion of the dam (Fig. 22), a pressure will be exerted, equivalent to a head of $z = 147$ ft. of water, or if $w = 0.03125$ tons per cu. ft., to an equivalent pressure of $147 \times 0.03125 = 4.6$ tons per sq. ft.

The Direct Pressure Impulse.—Assume that in Fig. 23, $v = f\,(t)$ represents the increase in velocity of the piston (or of the dam) during the acceleration period from $v = 0$ to a maximum value of $\sqrt{v_0}$. The pressure equation will be:

$$\frac{p}{w} = \frac{c}{g}\,v = \frac{c}{g}\,f\,(t) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (105)$$

[31] *Revue de Mecanique*, 1904; also "Hydraulique," 1923 Edition, p. 478; see, also "Introduction to the Study of Variable Flow" (in Russian), by Boris A. Bakhmeteff, M. Am. Soc. C. E., 1915.

In other words, the pressure curve will be identical in shape with the velocity curve, and the scale ratio will be $\frac{p}{v} = 4.6 \frac{\text{ton}}{\text{ft.}^2} \times \frac{\text{sec.}}{\text{ft.}}$.

The instant of maximum pressure coincides with that of maximum velocity, and is equal to $4.6 \times v_0$. Assuming sinusoidal motion with a maxi-

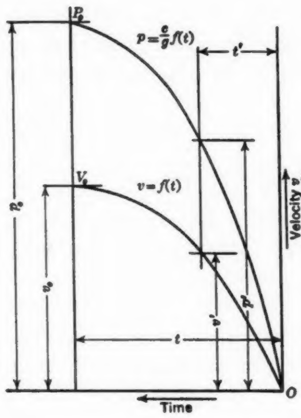


FIG. 23.

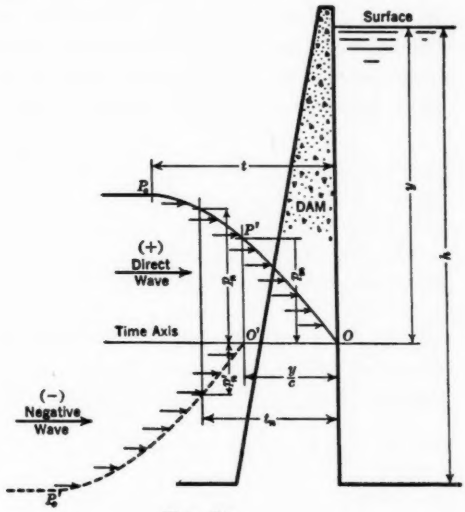


FIG. 24.

mum acceleration, αg, at $t = 0$, the maximum velocity will occur at $t = \frac{T}{4}$, and its value will be expressed by the equation:

$$v_0 = \alpha g \frac{T}{2\pi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (106)$$

Assuming that $T = \frac{4}{3}$ and $\alpha = 0.1$, as in the paper by Professor Westergaard, $v_0 = \frac{32.2}{10} \times \frac{\frac{4}{3}}{2\pi} = 0.68$ ft. per sec., and, therefore, the maximum pressure equals $p_0 = 4.6 \times 0.68 = 3.13$ tons per sq. ft., which is identical with values determined by the use of Equation (14).

In this discussion, the term, "direct wave," is defined as a pressure wave caused by horizontal motion only. Such a wave is said to be in direct phase. Regardless of the form of the velocity curve in Fig. 23, the maximum pressure in direct phase is determined entirely by the value of v_0 and is not affected by the length, τ, of the acceleration period.

Effect of Free Surface; Indirect Phase.—On the other hand, the acceleration period (and, therefore, the manner in which the displacement is imparted to the dam) is decidedly important when one considers the effect of the free surface of the water, or when the phenomenon is in the "indirect" phase.

In the case of water-hammer, the increase in pressure characteristic of the direct phase, is counteracted by the effect of a negative wave reflected from the direct positive wave at the open end of the penstock. The condition in general is identical to that of sound waves at the open end of organ tubes. By way of analogy, the effect of the open surface of the water behind the dam may be visualized as creating a negative pressure impulse, identical in form and coincident in phase with the direct pressure wave, but of opposite sign. This negative impulse, which will account for the vertical displacement of the water layers, will be propagated downward with the accoustical velocity, c, and will reach a layer at a depth, y, in a period equal to,

$$t = \frac{y}{c} \text{ sec.} \qquad (107)$$

measured from the time the dam first began to move (see Fig. 24).

At the surface ($y = 0$), where the direct and the negative impulses are generated simultaneously and where their phases coincide, the resultant pressure will always be equal to zero. At a depth, y, conditions may be visualized as shown in Fig. 24. The pressure caused by the direct wave which is generated by the movement of the dam is represented by the curve, OP_0. The negative impulse, $O'P'_0$, is identical in shape with OP_0, but opposite in sign, and has a difference of phase $OO' = \frac{y}{c}$. Obviously, while $t < \frac{y}{c}$, the effect of the surface yield has not had time to reach the layer at a depth, y, and the pressure increase is entirely due to the direct wave. For $t > \frac{y}{c}$, the resultant pressure is the difference between the ordinates of the direct and the negative wave. Thus, for $t = t_n$, the resultant pressure is $p_n - p'_n$.

If the shape of the velocity curve is as given in Fig. 23 (that is, if the acceleration is maximum at the beginning of the motion and then gradually becomes less), the maximum pressure at a layer, distant y from the surface, will be reached at the moment, $t = \frac{y}{c}$. This is the moment when the negative wave reaches the water layer under consideration. The effect of the acceleration period, τ, is now apparent: If it is small, the time, $\frac{y}{c}$, may be greater than τ, so that the negative impulse will not reach the layer in question in time to relieve the direct wave. In that case, the maximum pressure will attain its greatest possible value, $p_0 = 4.6\ v_0$, which corresponds to the direct impulse. A depth or an equivalent height of a dam may be determined for each value of τ by the equation,

$$h_0 = c\tau \qquad (108)$$

beyond which the increased pressure during the initial acceleration period will not be affected by the surface yield and the maximum pressure will be

fully determined by direct impulse. If $c = 4\,715$ ft. per sec., the values of h_0 for corresponding values of τ are as follows:

Values of τ, in seconds	Values of h_0, in feet
0.01	47
0.05	235
0.10	471
0.20	942

For sinusoidal motion, with $\tau = \frac{T}{4}$ and $T = \frac{4}{3}$ sec., $h_0 = 4\,715 \times \frac{1}{4} \times \frac{4}{3}$ $= 1\,570$ ft. This is in excess of the height of any dam thus far built. Therefore, it may be concluded that in actual structures the pressure never reaches the maximum value corresponding to the direct pressure impulse. This conclusion, however, will obtain only if the movement of the dam appears as represented in Fig. 25, in which the displacement begins at $t = 0$ from the extreme position and the velocity reaches its maximum over a period, $\tau = \frac{T}{4}$. On the other hand, conditions would be decidedly different if the motion was generated as shown in Fig. 26 by a sudden impact, such as in a ballistic pen-

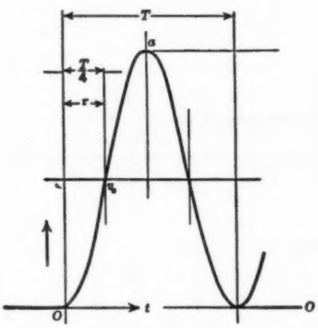


FIG. 25

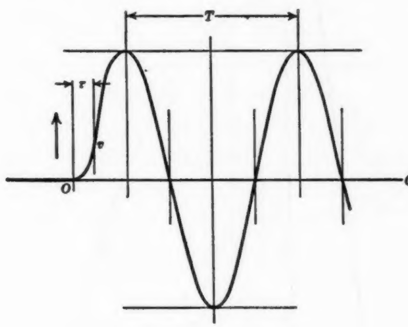


FIG. 26

dulum, and in which the maximum velocity is reached over a short initial impact period, which is brief in comparison with T, the period of the subsequent oscillations. At least some seismographic records seem to indicate movement of this character.[33]

Case of Sinusoidal Movement Shown in Fig. 25.— To compare data obtained by approximate reasoning outlined in the foregoing discussion, with the results of the exact method introduced by Professor Westergaard, assume the motion to be as in Fig. 25 with $T = \frac{4}{3}$ and $v_0 = 0.68$ ft. per sec., which corresponds to a maximum acceleration of 0.1 g. Assume that the maximum pressure, P_m (y), at a depth, y, will correspond to the direct impulse at the

[33] See, for example, the article on "Earthquakes," in the Encyclopedia Brittanica.

moment, $t = \frac{y}{c}$; and that further in view of Equation (108), $\frac{T}{4} = \frac{h_0}{c}$; then,

$$P_{m(y)} = P_0 \sin 2\pi \frac{\frac{y}{c}}{T} = P_0 \sin \frac{\pi}{2} \times \frac{y}{h_0} \quad \ldots\ldots\ldots\ldots (109)$$

If $P_0 = 3.13$ tons per sq. ft., and $h_0 = 1\,570$ ft., the value of the maximum pressure, P_m, for different heights of the dam, h, is as shown in Table 4.

TABLE 4.—VALUES OF MAXIMUM PRESSURE, P_m, FOR DIFFERENT VALUES OF h.

h, in feet	$\frac{h}{h_0}$	$\frac{\pi}{2} \times \frac{h}{h_0}$	$\sin \frac{\pi}{2} \times \frac{h}{h_0}$	P_m, in tons per square foot	P_w, in tons per square foot
(1)	(2)	(3)	(4)	(5)	(6)
200	0.127	0.200	0.197	0.62	0.47
600	0.382	0.600	0.565	1.77	1.51
800	0.510	0.800	0.718	2.24	2.18

Column (6), in Table 4, contains the values obtained by Professor Westergaard's equations for comparison. The pressures computed from Equation

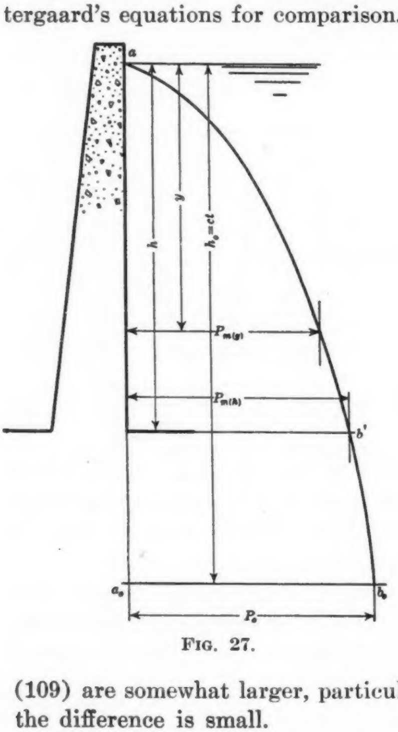


FIG. 27.

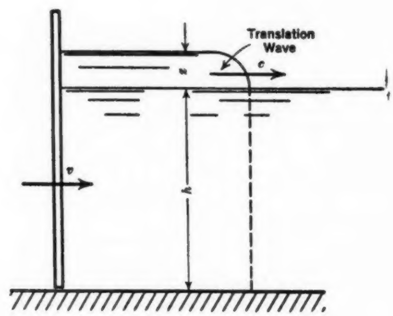


FIG. 28.

(109) are somewhat larger, particularly for small values of h. For high dams the difference is small.

Maximum Shear at Bottom.—In computing the total shear, Q, at the base of a dam, the effect of the negative impulse beyond the point, o', in Fig. 24 may be neglected, and instead the simple assumption may be made

that the shear is the sum of the pressures as determined for each depth layer by Equation (109). In this case (see Fig. 27):

$$Q = \int_0^h P_{m(y)} dy = \int_0^h P_0 \sin\left(\frac{\pi}{2} \times \frac{y}{h_0}\right) dy$$

$$= (p_0 h) \frac{2 h_0}{\pi h}\left[1 - \cos\frac{\pi}{2} \times \frac{h}{h_0}\right] = \beta' (P_0 h) \ldots\ldots\ldots\ldots (110)$$

Let $h_0 = 1\,570$ ft. and $P_0 = 3.13$ tons per sq. ft., then values of Q ars as listed in Table 5.

TABLE 5.—COMPARATIVE VALUES OF Q

h (1)	$\cos\frac{\pi}{2} \times \frac{h}{h_0}$ (2)	β (3)	$P_0 h$, in tons (4)	Q, in tons (5)	Q_w, in tons (6)
200	0.980	0.100	626	62.6	68.4
600	0.825	0.292	1 878	550	658.3
800	0.697	0.379	2 504	950	1 253

The shears in Column (5), of Table 5, obtained from Equation (110), are somewhat less than those computed by Professor Westergaard, which are given in Column (6).

Simplified Solution with Constant Acceleration.—The period, $t = \frac{y}{c}$, during which the pressure impulse is applied in the direct phase, is generally quite brief and, therefore, the assumption could be made that during this entire period the acceleration remains constant and retains its maximum value as taken from seismographic records and expressed by the product, αg.

With this simple condition no further assumptions are needed regarding the curve, $v = f(t)$, in Fig. 23. Obviously, this assumption increases the margin of safety in computing the stability of the dam. Constant acceleration denotes linear increase of velocity over the section op', in Fig. 24. As $t = \frac{y}{c}$, the velocity will be:

$$v = \alpha g \frac{y}{c} \ldots\ldots\ldots\ldots (111)$$

and thus the maximum pressure, $P_{m(y)}$, at the depth, y, will be expressed by the equation:

$$P_{m(y)} = v\left(\frac{c}{g} \times w\right) = \alpha\, w\, y \ldots\ldots\ldots\ldots (112)$$

which, with $w = 0.03125$ tons per cu. ft., gives, for the pressure at the base of the dam,

$$P_h = 0.031\ \alpha\, h \frac{\text{ton}}{\text{ft.}^2} \ldots\ldots\ldots\ldots (113)$$

The shear, in tons, at the base of the dam, is expressed by,

$$Q = \frac{P_h \times h}{2} = \frac{w}{2}\ \alpha\, h^2 = 0.016\ \alpha\, h^2 \ldots\ldots\ldots\ldots (114)$$

and the moment, in foot-tons, is expressed by,

$$M = Q \times \frac{h}{3} = \frac{w}{6}\,\alpha\, h^3 = 0.0052\,\alpha\, h^3 \quad \ldots\ldots\ldots\ldots (115)$$

Let $\alpha = 0.1$, then representative values of shear and moment are indicated in Table 6.

TABLE 6.—VALUES OF SHEAR AND MOMENT FROM EQUATIONS (110) AND (111)

h	P_h, in tons per square foot	Q, in tons	M, in foot-tons
200	0.62	64	41.6×10^3
600	1.83	575	$1\,123 \times 10^3$
800	2.48	1 022	$2\,662 \times 10^3$

The numerical coefficients in Equations (113), (114), and (115), should be compared with Items 9 and 10 in Table 2 of Professor Westergaard's paper.

The comparison of values computed by the approximate method and by equations presented by the author are, as follows:

	Approximate solution	Westergaard solution
$\frac{P_h}{\alpha h}$	0.031	0.0255 — 0.0296
$\frac{Q}{\alpha h^2}$	0.016	0.017 — 0.0198
$\frac{M}{\alpha h^3}$	0.0052	0.0068 — 0.0079

The Surface Surge.—It might be desirable to examine the nature of the phenomenon that occurs at the surface behind the dam. If a water-tight barrier (see Fig. 28), formerly at rest in a rectangular canal, were to be set in motion with a uniform velocity v, a surge or a wave of translation would be formed with a speed,[33]

$$c = \sqrt{gh}\,\sqrt{1 + \frac{3}{2} \times \frac{z}{h} + \frac{1}{2}\,\frac{z^2}{h^2}} \quad \ldots\ldots\ldots\ldots (116)$$

The height of this wave when v is not too large compared with c, will be:

$$z = h\,\frac{v}{c}\left(\frac{1}{1 - \frac{v}{c}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (117)$$

For small values of $\frac{v}{c}$ and $\frac{z}{h}$, it is permissible to assume that $c = \sqrt{gh}$ and $z = h\frac{v}{c}$, in which, c is the Lagrange velocity of propagation of waves

[33] St. Venant, Comptes rendus de l'Academie des Sciences, 1870; see, also, Bakhmeteff, "Hydraulics of Open Channels," 1932, p. 255.

of small height in a deep channel. For various depths, h, the values of c are, as follows:

h, in feet.	$c = \sqrt{gh}$, in feet per second.
200	80
600	139
800	160

In other words, for a dam 600 ft. high the surge created by an earthquake on the surface of the reservoir will travel with a speed of approximately 95 miles per hour. The velocity of the dam body itself, $v_o = 0.68$ ft. per sec. in the preceding examples, is insignificant by comparison. For a velocity $v_o = 0.68$ ft. per sec., the height, z, of the surge wave would be as follows (from Equation (117):

h, in feet	$\frac{v_o}{c}$	$z = h\frac{v_o}{c}$, in feet.
200	0.85×10^{-2}	1.70
600	0.49×10^{-2}	2.94
800	0.425×10^{-2}	3.40

This indicates the height of the surface wave that would be generated if the acceleration of the dam were slow and the velocity, $v_0 = 0.68$, was attained over a comparatively long period of time. The additional pressures in this case, caused by the rise of the water surface, would be the increase in hydrostatic pressure corresponding to the surge height, z. These additional pressures are quite insignificant when compared with the effect of the compression impulse considered in the foregoing analysis. The conclusion to be drawn is, that even with comparatively slow oscillations, $(T = \frac{4}{3})$, the pressures caused by earthquakes on dams are principally the result of elastic impulses within the water, and that the effect of the actual bodily displacement of the fluid is of secondary importance.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ANALYSIS OF CONTINUOUS FRAMES BY DISTRIBUTING FIXED-END MOMENTS

Discussion

BY HARDY CROSS, M. AM. SOC. C. E.

HARDY CROSS,[149] M. AM. SOC. C. E. (by letter).[149a]—The writer is indebted to those who have been kind enough to discuss the paper and have added so much to its value by illustrating the application of the principle involved.

Mr. Pilkey thinks the paper too brief. In the Seventeenth Century, Pascal wrote: "I have only made this letter rather long because I have not had time to make it shorter." (Provincial Letters, December 14, 1656.) The writer took time to make the paper short; as he looks at the mass of technical literature on his desk, he sincerely hopes the idea becomes popular. He thought best to confine the paper strictly to the fundamental principle and method involved, and not digress into the unlimited variations of this method or the unlimited applications of it. Secondary stresses, wind stresses, Vierendeel girders, continuous arches on elastic piers are special problems; while the elementary procedure is applicable to them, certain modifications are advantageous. The writer has indicated subsequently some of these modifications, but he has not felt justified in elaborating them in this closure.

Questions raised in some of the discussions have been clearly answered in others. The questions of Mr. Lyman have been answered in detail by Professors Martel, Morris, and Witmer, and Messrs. Pilkey, Black, Wessman, and Wilson. The limitations of the method suggested by Mr. Nielsen have also been discussed by others.

NOTE.—The paper by Hardy Cross, M. Am. Soc. C. E., was published in May, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1930, by Messrs. C. P. Vetter, L. E. Grinter, S. S. Gorman, A. A. Eremin and E. F. Bruhn; October, 1930, by Messrs. A. H. Finlay, R. F. Lyman, Jr., R. A. Caughey, Orrin H. Pilkey, and I. Oesterblom; November, 1930, by Messrs. Edward J. Bednarski, S. N. Mitra, Robert A. Black, and H. E. Wessman; January, 1931, by Messrs. Jens Egede Nielsen, F. E. Richart, and William A. Oliver; February, 1931, by Messrs. R. R. Martel, and Clyde T. Morris; March, 1931, by Francis P. Witmer, M. Am. Soc. C. E.; May, 1931, by Messrs. T. F. Hickerson, F. H. Constant, W. N. Downey and E. C. Hartmann; September, 1931, by Messrs. Thomas C. Shedd, David M. Wilson, and Marshall G. Findley; November, 1931, by Messrs. George E. Large, and Sophus Thompson and R. W. Cutler; January, 1932, by Alfred Gordon, Assoc. M. Am. Soc. C. E.; March, 1932, by Messrs. A. W. Earl, A. Floris, I. M. Nelidov, E. A. MacLean, George M. Dillingham, and Donald E. Larson; and April, 1932, by J. A. Van den Broek, Assoc. M. Am. Soc. C. E.

[149] Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

[149a] Received by the Secretary April 25, 1932.

Mr. Osterblom's discussion is interesting and valuable, especially his explanation of the fundamental simplicity of Maxwell's method and of the difficulties which arise in applying it.

The discussions of Professor Shedd and Mr. Oliver call for little special comment. The writer appreciates them and heartily concurs in Professor Shedd's remarks with regard to formulas.

With the discussion of Mr. Vetter, the writer differs in practically every statement. Mr. Vetter seems to have some serious objection to the method, but he does not make it clear. His interest in the vertical reactions at F and G is entirely academic. Of course, the total vertical reaction at C may be found by statics. This reaction is distributed between CF and CG in proportion to the "vertical stiffness" of these paths if "vertical stiffness" is defined as the vertical force at C along either path necessary to produce unit vertical deflection at C. Presumably it all goes to CG, unless CF is a "sky-hook." If the members are of constant section and immovable at G and F, the vertical stiffness is $\frac{1}{A\,E\,L}$, and it should involve no difficulties if the section varies.

The effect of shearing distortions is not involved in the problem of the distribution of the total vertical reaction at C between the points, F and G. In the paper the writer makes no assumption at all as to the effect of shearing distortions; if they are taken into account, the fixed-end moments, the stiffness values, and the carry-over factors are different from those used if these distortions are neglected. Of course, they should be neglected except perhaps in the most unusual cases. In the rare cases in which their effect in beams should be included, one should be guarded in accepting the conventional mechanics of internal stress in beams.

Whether longitudinal distortions shall be neglected depends on circumstances. It is a simple matter to correct for the effect of shearing and of longitudinal distortions by the method indicated in the paper. After the end moments are found in the beams, one computes the shears and the longitudinal forces and from these the distortions produced. He then finds that the frame is distorted by these displacements so that discontinuities would exist. The fixed-end moments needed to eliminate such discontinuities are then applied and these moments distributed throughout the frame. Theoretically, the procedure should be repeated, but except in the most unusual case, one trial will show the complete futility of the computation.

Mr. Vetter thinks that the engineer should "hitch his wagon to a star." He would do better to keep his feet on the ground. The conception of "mathematical accuracy" of the man on the street is decidedly amateurish; what the engineer needs is to get an answer with the requisite degree of accuracy without a prohibitive amount of labor.

The idea held by Mr. Vetter, that in some way (which he does not make clear), the method of moment distribution is approximate and involves assumptions which the theory of least work and the method of slope deflection escape, shows a fundamental misconception of these methods. Any one who

computes the moments in a frame of reinforced concrete will make many assumptions as to the properties of materials, conservation of plane sections, and other matters; E has no definite value and I has no definite meaning in the analysis of frames of reinforced concrete. This is true by whatever method the frame is analyzed. Whether the computer knows that such assumptions are made is quite another matter.

Mr. Vetter seems also to object to the writer's list of incidental problems involved and states that he could add a great many more, "which have just as much, or more, to do with the subject under discussion [moment distribution?]" That is excellent, or it would have been excellent had he done so; one of the things needed in connection with continuous frames is a clear recognition of the complications involved. Another need is for a simple method of studying the effect of such complications; moment distribution is such a method. The three samples of complications furnished by Mr. Vetter are not of the type listed by the writer. Change in length of members due to shrinkage and to change of temperature and the settlement of supports are clearly recognized phenomena, the effect of which, for given data, are easily evaluated by moment distribution exactly as indicated in the text, thus:

> "Imagine any joint in a structure, the members of which are being deformed by loads, or in some other way, to be first held against rotation and then released."

The method is not restricted to analysis for loads on the structure.

Mr. Eremin thinks that,

> "In the limitations of his method, the author omitted to state that the modulus of elasticity of materials was considered to be constant for all members of the frame and also that the lateral rigidity of the members of a continuous frame is sufficient to prevent buckling."

The writer thinks that the first statement is in error. One does not necessarily assume either that E is constant for all members or from section to section in any one member or over any one cross-section of the member. It is merely necessary to have an accurate stress-strain diagram for the material. If it is first assumed that E is constant and all stresses are computed on this basis, the fixed-end moments, stiffness values, and carry-over factors may then be revised for the values of E indicated by these stresses. The operation may be repeated to any degree of precision. The problem is evidently one for research, not for the office. The writer has studied it in relation to the effect of chance variation of the modulus of elasticity,[150] the effect of plastic flow and time yield in concrete, and the effect of secondary stresses beyond the elastic limit of steel. The common assumption of the constant, E, is a matter of convenience, and is not a limitation of method.

The statement that the method is restricted by the assumption that the members do not buckle seems to the writer to mean that the method will give the moment in a member if the member is still there—if it has not failed. This is correct.

[150] "Dependability of the Theory of Concrete Arches," *Bulletin 203*, Eng. Experiment Station, Univ. of Illinois.

Mr. Mitra deplores the fact that the method does not lead to any general equation. Since the avoidance of such general equations was the object of the paper, the writer is pleased by the comment. That the method of slope deflection and the graphical method of conjugate points are more accurate is not true; that these methods are quicker and easier to use is opposed to the experience of hundreds of engineers in the United States.

Professor Hickerson evidently does not like the paper at all, even though he "commends the author." Consequently, he introduces the subject of "Tabular Coefficients." Of course, it is better to use the answer previously determined for a problem than it is to re-work the problem, provided one is sure that the answer is correct and provided the answer is easily found. The trouble with tabular coefficients is that one cannot be sure, unless he checks them, that they are correct, however distinguished their author, and that the one using them may not agree with the basis on which they are computed. Unfortunately, few published tables explain clearly the basis on which they are prepared. Even so careful a scholar as Professor Hickerson fails to state whether the negative moments which he has computed are at the face of the support or at the center, and what allowance, if any, he has made for the increased flexural resistance of the members between the faces of any support.

Professor Hickerson gives a coefficient for an arch frame uniformly loaded between columns. He then states that "if the entire load is concentrated at the central ridge, * * * the foregoing values would be multiplied by $\frac{3}{2}$." This is not quite correct either here or in general. In this case it is nearly true but any one using the tables must determine for himself when it ceases to be approximately true. Thus, any one using the "tabular coefficients" will still need a method of analysis.

Mr. Floris thinks that the idea of using fixed-end moments in the analysis of a continuous frame is not new. Of course it is not, but Mr. Floris has gone far afield for his references. Fixed-end moments have been used in slope deflection in this country for years and apparently by Manderla before that and probably by others before Manderla. Mr. Floris, moreover, has overlooked the paper presented by Professor W. M. Wilson at the World Engineering Congress at Tokyo. The book in Japanese is not available to the writer and would do him little good if it were. The method of the late Professor Ostenfeld referred to by Mr. Bednarski and by Mr. Floris is apparently an extension of slope deflection along the lines indicated by the writer in his monograph on the column analogy.[151] The presentation by Mr. Bednarski is, unfortunately, not very impressive. The writer fails to find anything like the method of the paper in Hartmann's "Statisch Unbestimmte Systeme."

Two conventions of sign are possible for moments at the ends of the members of a frame. The convention of rotation for statical moments is indicated where rotations are involved in the solution and, therefore, is used in the method of slope deflection. The alternative convention is for bending

[151] "The Column Analogy," *Bulletin 215*, Eng. Experiment Station, Univ. of Illinois.

moments. Where a structure is to be designed from the moments found by the analysis the writer still feels that it is best to adopt throughout the analysis the convention commonly used in design. Several who have preferred the convention of rotation have used it in connection with problems of wind stress or of secondary stress. In such problems the writer also prefers the convention of clockwise rotation, for with wind stresses and secondary stresses it usually makes no difference which side of a member is in tension. The object of analyses for secondary stresses is to determine whether they are too high, not to use them in design; the wind may blow in either direction. In any case, one should consistently follow one method, as Mr. Gorman notes.

Some misunderstanding has appeared in the discussions and some misstatement has appeared elsewhere as to the restriction given in the paper for the method. The writer does not confine the method to frames in which the joints do not move, but merely states that these movements must take place in finite jumps so that "the joints do not move during the process of moment distribution." The method is applicable, without special complication, to all frames for any condition of loading, transverse as well as vertical, and for any condition of deformation, whether due to shrinkage, change of temperature, or settlement of support. Several discussions illustrate such applications.

The first part of the paper is restricted to structures in which the joints do not move. The writer explained that if such movement actually takes place, allowance may be made for it as indicated later in the paper. This procedure has been explained in considerable detail by several contributors. With regard to this movement of the joint (referred to in the paper as side-sway), four questions are important: (1) Can side-sway occur in the structure? (2) Does side-sway occur for those conditions of loading which control the design of the structure? (3) Is this side-sway a matter of importance in design and is an approximate solution sufficiently accurate? (4) How shall one determine the exact moments produced by this side-sway?

In most structures side-sway produced by vertical loads only is either impossible or improbable. Thus, the probability of the lateral movement of a floor of a building due to vertical loads is negligible. Even where the forces act transversely, side-sway of individual bents is commonly either prevented or checked. Analysis for the moments produced by crane loads in a mill-building bent are often incorrect. If the load produces side-sway of each individual bent, there would occur, as the load moves, so much lateral vibration as to impair, seriously, the usefulness and the durability of the structure; such movement must be prevented or checked by some system of bracing and the bent then is not free to sway sidewise. Similarly, the bent of an elevated railway is not free to sway because of vertical live loads, but is restrained by horizontal bracing of the girders which frame into or rest upon the bent. The writer has also seen solutions presented for stresses produced in lateral frames of bridges due to unsymmetrical loading of the floor-beams which neglected the restraint against relative movement of the planes of the top and bottom chords of the bridge offered by lateral bracing.

In those cases in which side-sway due to the vertical loads is possible, it is frequently most improbable. An unsymmetrical vertical load may indi-

cate lateral movement of any one loaded bay in a building of reinforced concrete, but for such lateral movement to occur freely it would be necessary for the entire floor to be loaded in this same way; the probability is that lateral sway will not occur. Attention should be called to the fact that side-sway is often a mitigating influence tending to decrease the maximum stress in certain parts of the structure, especially in the more highly stressed column; the assumption of side-sway where it does not exist may be on the side of danger.

In those cases in which side-sway may occur, it is neither necessary nor desirable to take account of it for each condition of loading. It is more convenient to correct for it only for those combinations of loading which produce maximum controlling moments.

Side-sway is much like the effect of wind. If a frame is analyzed on the assumption that the joints do not move and movement of the joints will actually occur, it will be found that the sum of the shears in the columns is not equal to the known shear. Movement of the joints must then have been prevented by forces acting at these joints. These forces reversed in direction may then be treated as wind loads. Moments may then be found by some of the approximate methods of wind-stress analysis to determine how large are the stresses involved.

In many cases side-sway does not exist; in other cases, it is of no importance; in most of the remaining cases. it is sufficiently unimportant to justify the use of approximate methods of wind-stress analysis without hesitation. In the remaining cases in which it requires precise analysis an indirect method may be used.

"Assume any series of fixed-end moments in the legs such that all legs have the same deflection. * * * Distribute these fixed-end moments and find the total shear in the legs." Add these moments to those assuming no side-sway in such ratio that statics is satisfied. The subject has interested several contributors. The procedure may be by direct proportion as indicated by the writer in another paper,[152] and as shown by several discussers; it may be by finding first the effect of unit shear in one story at a time and combining the solutions by a method of successive convergence, as suggested by Professor Grinter; one may write a series of formal equations, as Professor Constant suggests; or one may use successive approximations to the true values, as indicated in detail by Professor Morris.

It does not seem to the writer that any one of these variations of technique has any general advantage; sometimes one and sometimes another is the more useful. Thus, in a single-story frame, it is undoubtedly more expeditious to find the effect by direct proportion, but in a high building the method used by Professor Morris offers some advantages.

Another variation of method saves time in analyses for wind stresses. The joints are not assumed to be fixed-ended, but it is assumed that all joints not fixed have the same rotation, the moments in the girders being proportional to their K values and the moments in the columns, due partly to rotation of

[152] "Continuity as a Factor in Reinforced Concrete Design," by Hardy Cross, *Proceedings*, Am. Concrete Inst., 1929.

the joints and partly to displacement of the floors, being those determined by the shears. If the ratio of girder moments to K values is chosen with discretion, all the joints may often be nearly balanced before any moments are distributed. In the columns in the first story an adjustment must be made for the fixation at the base.

Fig. 75 shows the revised computations for the building discussed by Professor Morris. Only one distribution and one shear adjustment have been made. The moments given by Wilson and Maney are also given on the diagram (in brackets, thus: [24.0]); it will be seen that the agreement for girder moments is much more satisfactory than in Professor Morris' solution. Too many conclusions should not be drawn from this case, however; the building is too regular to be taken as typical of those variations of framing arrangement which present, in modern design, the real problems of wind stress.

In Fig. 75, two devices have been used to simplify the procedure. In the first place certain moments proportional to K have been thrown arbitrarily into the girders so that to begin with, all joints are nearly balanced. These represent the effect of equal rotations at all joints except the bases of the columns; this exception is provided for in the columns of the bottom story by use of the factor, $\frac{1}{C}$, discussed later.

In the second place, the analysis is abbreviated by writing only the moments carried over and making one distribution at the end of the procedure. The moments produced by each distribution which would need adjustment to balance the shears are evidently equal to three times the moment carried over in the columns in any story.

The solution is presented here as an interesting variation of the technique of moment distribution and not necessarily as a procedure recommended for the analysis of wind stresses in all cases.

A valuable by-product in wind-stress analyses is the direct determination of deflections. Each addition of fixed-end moments in any column represents a relative deflection of the floors at the ends of that column equal to $\frac{Mh}{6\,EK}$, in which, M is the fixed-end moment at one end of the column. In the solution just given the total moment added in a column is due partly to rotation and partly to displacement. That due to rotation is $\left(\frac{K}{2} + \frac{K}{10}\right) = 0.6\,K$ in columns above the first story. Hence,

$$\Delta = \left(\frac{M}{6K} + 0.1\right)\frac{h}{E}$$

in stories above the first, in which, M is the total added moment at one end of the column. In the first story,

$$\Delta = \frac{Mh}{6\,EK}$$

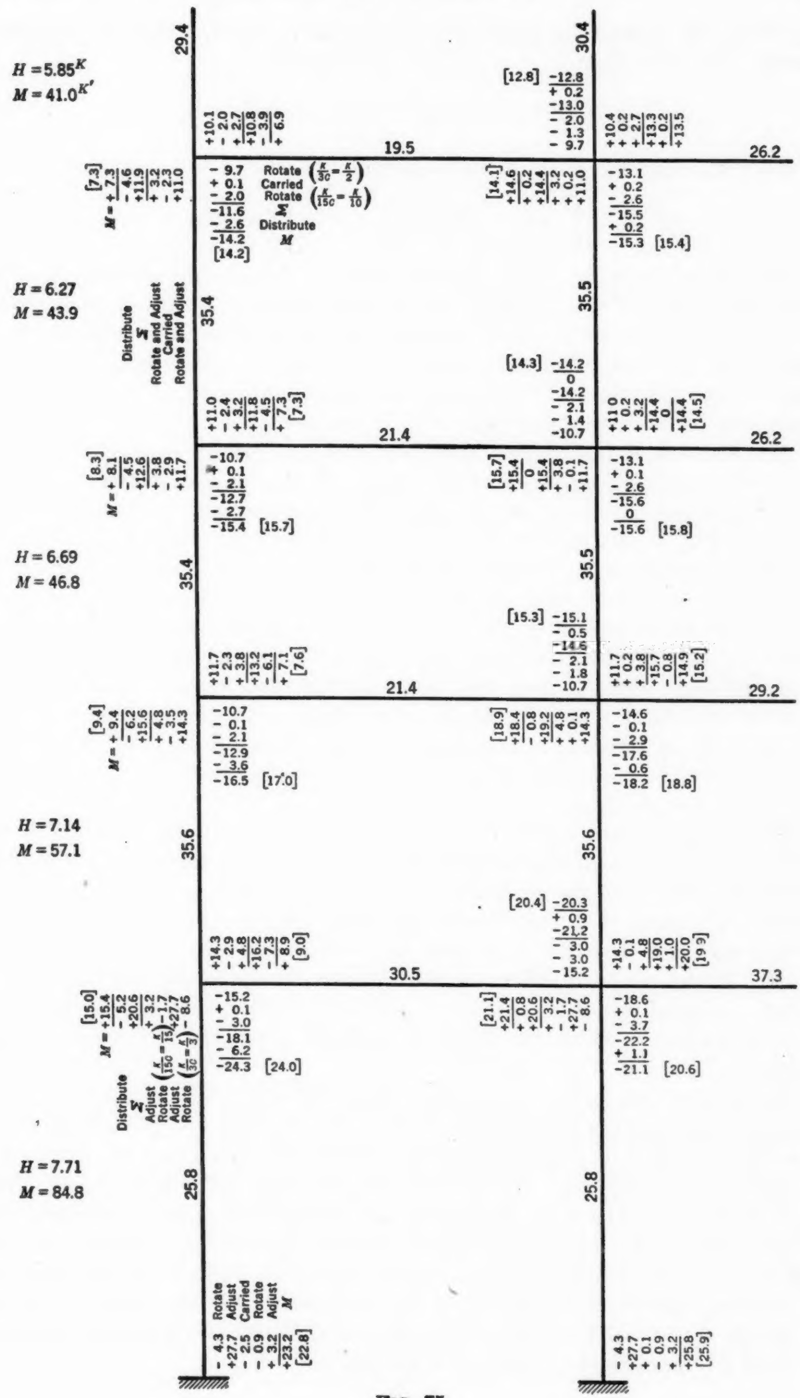

FIG. 75

in which, M is the total adjusting moment at one end, which is given separately. Thus, is the first story, in the wall column,

$$\Delta = 12 \ (27.7 + 3.2) \ \frac{h}{6\ EK} \ = 0.0211 \text{ in.}$$

In the second story, in the wall column,

$$\Delta = 12 \left[(14.3 + 4.8) \frac{1}{6K} + 0.1 \right] \frac{h}{E} = 0.0146 \text{ in.}$$

The frame analyzed by Mr. Nelidov offers an interesting illustration of the wide application of the method. The problem however, is solved more readily by the column analogy, as are probably all cases of an arch or bent of one span. The method of moment distribution has the advantage, however, of raising some question as to whether horizontal movement at B, C, and D is restrained by horizontal bracing, as it should be.

Mr. Nelidov divides frames into two classes according to the interrelation of possible linear movements of the joints. It is probably better to classify them according to the number of independent movements possible. Thus, the frame of Fig. 1 may be said to have no freedom of joint movement (assuming that D, say, is a fixed bearing); Fig. 5 has no joint freedom (A is fixed); also, Fig. 7 has one degree of freedom of joint movement; Fig. 8 has one; Figs. 9 and 10, two; Fig. 12, two; Fig. 22, one; Fig. 31 may well be treated as one span with no joint freedom (at A or A'); Fig. 32 has four degrees of joint freedom; Fig. 33 may well be treated as two spans with one degree of joint freedom; and Fig. 65 has two degrees of joint freedom. The degree of joint freedom is important in that it determines the number of simultaneous equations that must be solved in some way to satisfy the equilibrium of linear forces.

Mr. Bruhn suggests that the method has important applications in the field of airplane design. The writer knows that this is true, but is not sufficiently familiar with the problems involved in this field to illustrate its usefulness; he hopes that Mr. Bruhn will find time to do so. Certainly the method should facilitate the analysis of continuous struts subject to flexure. As Mr. Bruhn suggests, there is no difficulty in extending the method to include torsional effects. The writer has used the method in special problems of space frames in which the effect of torsional resistance is included in the analysis.

Mr. Downey has presented an interesting discussion of the application of successive convergence to the method of slope deflection. Slope deflection has more than justified its place in America and Mr. Downey has made a valuable addition to this literature. If, however, Mr. Downey will arrange his computation in the way shown by the writer and, further, if in using moment distribution one writes only the moments carried over in each member, it will be found that his computations parallel closely those of moment distribution; but in slope deflection the moments are first translated into rotations which are finally translated back into moments and this represents additional work not used in the method of moment distribution.

These relations are brought out by the problem shown in Fig. 76. This problem has been solved by slope deflection (Fig. 76(*a*)) and by moment distribution (Fig. 76(*b*)). As an additional check it has been solved also by the abbreviated method of moment distribution for a three-span beam in which the converging series are summed at once (Fig. 76(*c*)). By the method of slope deflection, using the method of successive convergence, the values of ϕ are first found. These values of ϕ are next converted into moments and added to the original end moments. In moment distribution the unbalanced

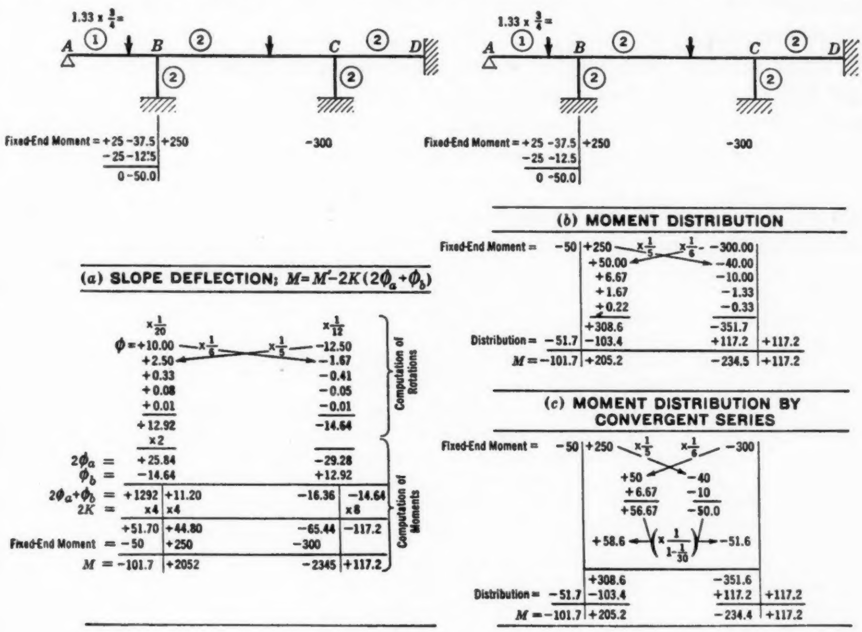

FIG. 76.

moments are carried over just as the ϕ values were carried over, the amount of computation being the same; but in moment distribution the quantities dealt with are always moments, which are the thing sought; by the method of slope deflection these moments are converted into rotations, from which the moments must be recomputed in the end.

Any frame must satisfy two general groups of conditions. The forces acting on it must satisfy the laws of statics so that the frame shall be in equilibrium. The forces acting on it must also satisfy the conditions of geometry, so that the rotations produced in the frame by these forces shall maintain continuity in the frame. The moments at the joints may be expressed either in terms of the joint rotations in the first place and these rotations found from the equations of static equilibrium at the joints, or the rotations may be expressed in terms of the moments and these moments found from the conditions of continuity at the joints. The latter method is that used in developing the theorems of three moments and of four moments. The former method

has been used in the United States by writers on slope deflection. The method of moment distribution clearly belongs in the class with the theorems of Clapeyron, but it should be obvious that all kinds of combinations of the equations of statical equilibrium and of geometry are possible, and that it is worse than futile to discuss whether a method introduces any principle in this field which is not implied in principles already known. There will be no new principles in this field, but by varying the form or arrangement of the equations and by more convenient methods of solution a clearer picture of the procedure of analysis can be found and the labor involved may also be reduced substantially.

What has just been written with regard to the theorem of three moments and the method of slope deflection is equally applicable to the method of work. The method of work is simply a deduction from elementary geometry.

Professor Richart suggests modification of the K values in certain stock cases. General relations to be used in direct distribution may be stated as follows. The moment needed to produce unit rotation at one end of a member may be written in terms of the physical properties of the member and the ratio of the changes in end moments in the member due to rotation.

Let M_a = change in moment from the fixed-end condition at the end, A, under consideration;
M_b = change in moment from the fixed-end condition at the other end of the member, B;
S_a = stiffness of the member at A, defined as the moment necessary to produce unit rotation at A when B is fixed;
r_a = carry-over factor at A (from A to B); and,
r_b = carry-over factor at B (from B to A).

Then, for unit rotation at A,

$$M_a = S_a \frac{1 - r_a r_b}{1 - r_b \dfrac{M_b}{M_a}} = S_a \frac{1}{C}$$

If, then, the ratio, $\dfrac{M_b}{M_a}$, is known for all members at a joint, the unbalanced moment at that joint may be distributed directly.

In certain cases the ratio, $\dfrac{M_b}{M_a}$, is known from inspection. When the member is fixed at B, $\dfrac{M_b}{M_a} = r_a$; when the member is free at B, $M_b = 0$; in cases symmetrical as to form and loading, $M_b = M_a$; and in anti-symmetrical cases, $M_b = -M_a$.

Values of the constant, $\dfrac{1}{C}$, are shown in Table 21. The general expression may be deduced, of course, in several ways; many will deduce it from area moments. This general relation leads also to a check on the analysis. It is easy to see whether the conditions of statics are satisfied by observing the

balance of the joints. The conditions of geometry are satisfied if $M_a - r_b M_b$ is proportional to $\dfrac{S_a}{1 - r_a r_b}$, or, in the case of beams of constant section, if $M_a + \frac{1}{2} M_b$ is proportional to K at any joint.

As one uses the method of moment distribution it soon becomes evident that it is not necessary to write the distributed moment each time. One may write only the moment carried over and make all distributions in one opera-

TABLE 21.—VALUE OF CONSTANT, $\dfrac{1}{C} = \dfrac{1 - r_a r_b}{1 - r_b \dfrac{M_b}{M_a}}$

Description of beam	Unsymmetrical haunching, $r_a >$ or $< r_b$	Symmetrical haunching, $r = r_a = r_b$	Uniform section, $r_a = r_b = -\frac{1}{2}$
Beam fixed at far end	1	1	1
Beam simply supported at far end	$1 - r_a r_b$	$1 - r^2$	$\frac{3}{4}$
Symmetrical	$\dfrac{1 - r_a r_b}{1 - r_b}$	$1 + r$	$\frac{1}{2}$
Anti-symmetrical	$\dfrac{1 - r_a r_b}{1 + r_b}$	$1 - r$	$\frac{3}{2}$
Beam continuous at far end	$(1 - r_a r_b)$ to 1	$(1 - r^2)$ to 1	$\frac{3}{4}$ to 1

tion at the end of the process. Time is saved, since almost one-half the figures are omitted, but the simple physical picture of the operation is considerably obscured.

This procedure may be carried further in any case where the value, $\dfrac{1}{C}$, is known for all spans but one in a frame. Carry over the proper proportion of the unbalanced moment in this span and repeat. Let a and b be the proportions carried over at the two ends. If the procedure of carrying over moments is continued it will be seen that they run to an infinite converging series the sum of which equals $\dfrac{1}{1 - ab}$ times the sum of the first two terms. Thus, the total moments carried over may be written after two terms and the total unbalanced moment may be distributed.

Fig. 76(*c*) shows the procedure. The unbalanced moments at B and C are carried over twice. The sum of the moments thus carried over is then multiplied by $\dfrac{1}{1 - ab} = \dfrac{1}{1 - \dfrac{1}{5} \times \dfrac{1}{6}}$. The sum thus found for the moments carried over is then added to the original fixed-end moments and the total is distributed at the joint.

This method of writing at once the sum of the converging series may be extended to frames in which the value, $\dfrac{1}{C}$, is unknown for more than one span in the frame. The general procedure of exact moment distribution by summing the convergent series is of great interest and, in some cases and for

some purposes, of great value; but such methods are not properly a part of a paper the object of which is to present a single general method involving no special paraphernalia.

It is interesting to note that to determine moments in any span any system whatever may be reduced with close approximation to an equivalent structure of three spans, such as *A B C D* in Fig. 76, by estimating or computing the fixation of the outer ends of all members except *BC*. Professor Van den Broek indicates the use of such substitute frames; so also does Professor Richart. The writer also used the concept of equivalent stiffness in his paper, "Continuity as a Factor in Reinforced Concrete Design," previously mentioned.

As another illustration, in the truss analyzed by Mr. MacLean the members meeting at *C* are known to remain fixed at the end at this joint; also, the member, *BD*, is known to be symmetrically distorted and $\frac{1}{C} = \frac{1}{2}$. The value, $\frac{1}{C}$, is then known for all members except the end posts. By twice carrying over distributed moments in this member and then summing the series by multiplying by $\frac{1}{1-ab}$, the total change in end moment due to carry over is at once determined and can be directly distributed.

In secondary stresses this method of convergence is usually only of academic interest. In the problem solved by Messrs. Thompson and Cutler it is not applicable, but the procedure used by them is much shortened if all distribution is made once for all at the end of the analysis.

Extension of the method of moment distribution to the study of the effect of gusset-plates on the secondary stresses is of some interest and presents no great analytical difficulty, the members being treated as of varying section. Any one who has analyzed on this basis will recognize more clearly the futility of great precision in computations on the usual basis.

Mr. Hartmann suggests that time may be saved by distributing unit unbalanced moments at the individual joints and then combining for any combination of loading. Mr. Dillingham uses the method in connection with moving load systems. This procedure has much merit where many combinations of loading are to be considered. It is not necessary, however, to distribute a unit moment in each member, but only at each joint. The distribution at each joint being known, any unbalanced moment from any of the members meeting at that joint may be distributed at once. The writer always uses this method where moving concentrated loads are under consideration. The fixed-end moments are found from influence lines. These are available for beams of varying section in a recent paper by Mr. L. T. Evans.[158] The unbalanced end moments are then distributed by the factors already found. On a string polygon for the load system, one then lays off the end moments at the supports, draws the closing lines in each span, and thus constructs the entire curve of moments for any position of the loads.

[158] "The Modified Slope-Deflection Method," by L. T. Evans, *Journal*, Am. Concrete Inst., October, 1931.

This procedure may be abbreviated by distributing moments at a support near the center of the structure. This joint is then held rigid while distribution is made at adjacent joints on either side, the resulting unbalanced moment at the first joint being then distributed from the values already found. Thus, the distribution of individual moments becomes more rapid as the ends of the structure are approached.

Fig. 77 shows the application of this procedure to the problem solved in the original paper. An unbalanced moment is first distributed at *B*. Then, when unbalanced moments are distributed at *A* and *C*, *B* is first treated as fixed and the values previously found are used to distribute the unbalanced

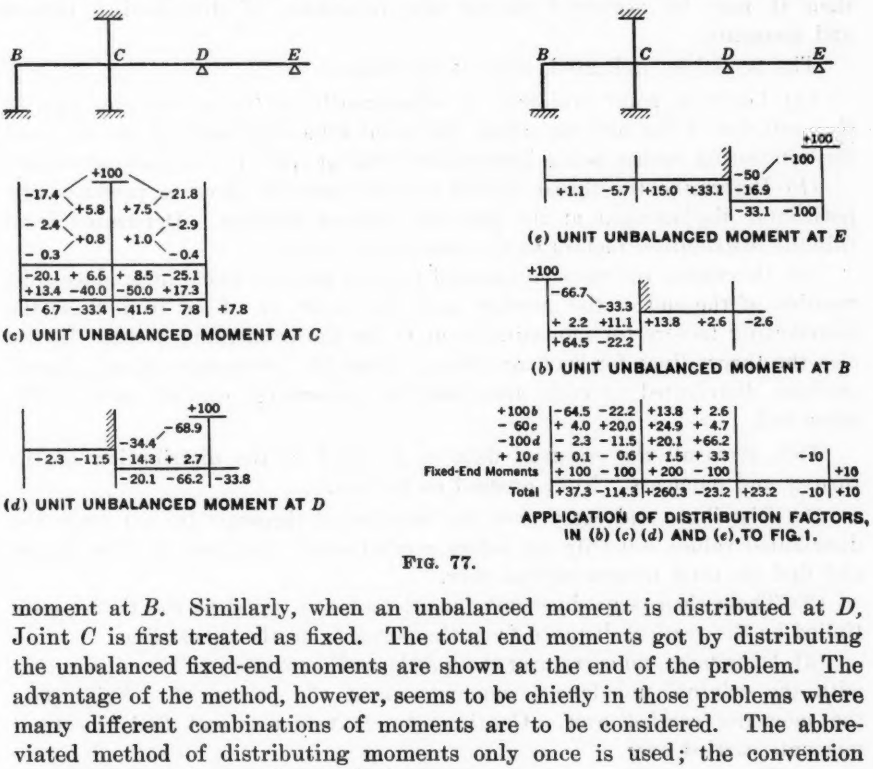

FIG. 77.

moment at *B*. Similarly, when an unbalanced moment is distributed at *D*, Joint *C* is first treated as fixed. The total end moments got by distributing the unbalanced fixed-end moments are shown at the end of the problem. The advantage of the method, however, seems to be chiefly in those problems where many different combinations of moments are to be considered. The abbreviated method of distributing moments only once is used; the convention of signs herein is that for statical moments.

Mr. Larson has made a valuable addition to the paper. The method of moment distribution is applicable to beams which are curved as well as to those which are straight, and is applicable to the distribution of forces as well as of moments. In order to use it conveniently, however, it is important that the convergence be rapid.

In the solution indicated by Mr. Larson it will be seen that the convergence is not very rapid. This is due to the large unbalanced thrusts and moments thrown back into the system each time by the pier. This disturbing factor may be avoided by distributing moments not about the pier top, but about

some point so chosen that when the moments are distributed by rotation of the pier top about this point, the resulting thrusts are also balanced. It will also be true that there will be no unbalanced moment produced about this point due to distribution of unbalanced thrusts.

If this is done the pier may then be neglected during the process of distributing thrusts and moments. Indeed, the side arches may be neglected also, since only those thrusts and moments are distributed that are carried over from adjacent joints, none being carried over from the fixed ends. If it is desired to treat either one of the end arches or a pier as hinged at the foundation, its elastic properties may be determined for this condition and then it may be neglected during the procedure of distributing thrusts and moments.

The procedure indicated, then, is as follows:

(*a*) Locate a point (referred to subsequently as *O*) on the pier axis so that rotation of the pier top about this point (the other ends of the pier and the connecting arches being immovable), will produce no unbalanced thrust.

(*b*) Determine the thrust needed in each member alone to produce unit horizontal displacement at the pier top without rotation. Determine from this the distribution factors to the connecting arches.

(*c*) Determine the moments needed in each member alone to produce unit rotation of the end of the member about the point, *O*. Find from these the distributing factors for moments about *O* for the connecting arches. Locate also the thrust lines for such rotations. From the percentage of unbalanced moment distributed to each arch find the percentage carried over to the other end.

This gives all the essential data to be used in the distribution of the thrusts and moments. Now, proceed as follows:

(1) Distribute and carry over the unbalanced thrusts. Do not write the distributed values, but only the values carried over. Continue to convergence and find the total thrusts carried over.

(2) These thrusts produce unbalanced moments equal to the thrust multiplied by the vertical distance from the point, *O*, to the thrust line.

(3) Distribute and carry over the unbalanced moments just found, together with the original unbalanced moments about the points, *O*. Write only the moments carried over. Continue to convergence and find the total moments carried over.

(4) These moments produce unbalanced thrusts equal to the moments divided by the vertical distance from *O* to the thrust line.

Repeat the cycle to any desired accuracy. Next, by addition, find the total thrusts produced by displacements and the total thrusts produced by rotations. Since their lines of action are known, any other quantities may be determined by statics.

Since it is now necessary to deal only with the arches during the process of distribution, a somewhat more convenient convention of signs for thrusts than that used by Mr. Larson may be adopted. If the usual convention of signs is followed for moments used in the design of arches and girders that

positive moment produces tension on the lower side, then a thrust acting above the axis—on the positive side—produces positive moment. Thrust in the arch is positive and tension negative. This makes it possible to add thrusts carried over to opposite ends of the arch without confusion.

Fig. 78 shows the solution of the problem given by Mr. Larson, using this technique. The percentages to be carried over and the lever arms of the thrusts have been determined from the data given by Mr. Larson. The final

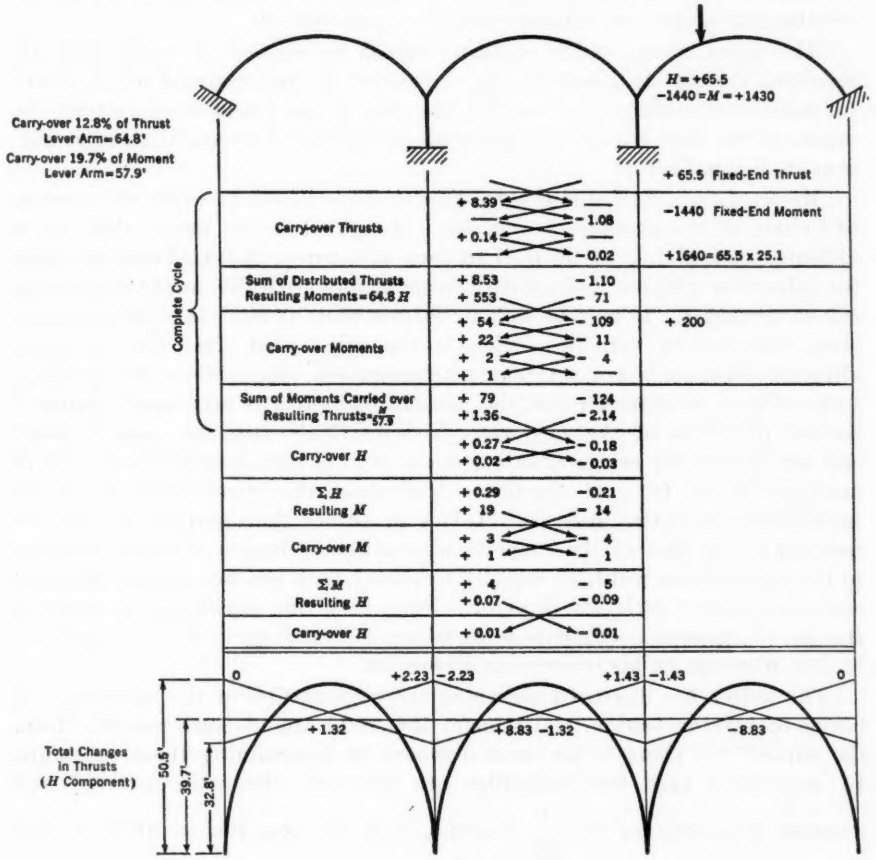

FIG. 78.

solution is obtained by adding the different thrusts and is indicated at the bottom of the diagram. The solution is shown here only to illustrate how, in this special case, a modification of the technique facilitates the convergence. The writer hopes to elaborate this procedure, elsewhere. It has a wide field of application in studies of continuous arch series in bridges or in buildings and of combinations of girders and arches.

It is not the function of this paper to discuss methods of deriving the elastic properties to be used in distributing the fixed-end moments and thrusts

to the piers and arches. The writer prefers to use the theorems of the column analogy, according to which, the stiffness values and lines of thrust are defined as follows: (1) The stiffness for thrust is the stress at unit vertical distance from the neutral axis of the analogous column section for unit moment about the horizontal axis of this section; (2) the stiffness for moment is the stress on the analogous column section at the center of rotation for unit load at this center; and (3) the lines of thrust are the neutral axes of the analogous column section for these loadings. These theorems are equally applicable whether the arches are symmetrical or unsymmetrical.

The point about which rotation should be applied in order that the resulting thrusts shall balance may be found as "the centroid of the values for horizontal stiffness," or so that the sum of the products of vertical distances to the thrust lines for displacement multiplied by the horizontal stiffness shall equal zero.

Messrs. Large, Earl, and Gordon have discussed cases in which the moment of inertia of the members is variable. As stated in the paper, there is no difficulty in applying the method to such structures; the fixed-end moments, the values for stiffness, and the carry-over factors are different if the members are of varying *I*. It may be well to observe that, at least in reinforced concrete, the case of variable section is the rule rather than the exception, although this fact is not yet generally recognized. Even where the members seem to be of constant section, the moment of inertia is very much increased beyond the faces of the supports. Note carefully that the sum of fixed-end moments at the center of intersections is to be distributed, not the sum of moments at the faces of supports. Neglect of this requirement of statics invalidates the entire analysis. Tables or curves that purport to give the moment at the face of the supports of continuous frames without reference to the ratio of the width of support to span length are necessarily incorrect and may be very seriously in error. Note, also, that traditional practice in the use of moment coefficients is based on the clear span, as is pointed out by Mr. Wessman in his interesting discussion.

The writer has discussed elsewhere the computation of the constants for beams of varying section, and thought it best to exclude such material from the paper.[151 152] There is no great difficulty in determining these constants. In any given case five quantities are involved, the area, centroid and moment of inertia of the $\frac{1}{EI}$ diagram, and the area and centroid of the $\frac{M}{EI}$ diagram, in which, M is from any trial moment curve. These quantities are then to be combined. There are evidently endless variations of detail in doing this, the principal rule being to follow a definite procedure and avoid repetition of computation. The writer prefers to use the column analogy which converts the entire procedure into the familiar process of computing fiber stresses in beams.

In his paper previously mentioned, Mr. Evans has given constants for members of varying sections. It will be of interest to those who use moment

distribution to note that the values for moment coefficients given by Mr. Evans are for fixed-end moments, the beam coefficients, C_1 and C_2, are for stiffness, and the carry-over factor at one end is $\frac{C_2}{C_1}$, and at the other end, $\frac{C_2}{C_3}$.

Curves and constants for the properties of haunched beams of reinforced concrete based on the assumption that I varies as the cube of the beam depth, however, should not be taken too literally. The effect of cracking, and the action of the flange in tension and in compression, introduce many uncertainties.

All kinds of ingenious tricks may be used to shorten the procedure. This is all right for the specialist who is engaged daily in this field. The general method presented in the paper originated from studies made by the writer in the analysis of three-span frames and is his final choice for ordinary daily use from many variations of method that he has developed and used. Any one can develop an unlimited number of such variations. Their novelty is no virtue; what is wanted for daily use is maximum simplicity consistent with reasonable facility. Such variations present a fascinating occupation to dilettantes who amuse themselves in the field of indeterminate structures; for the ordinary designer who analyzes such frames not as a vocation or for amusement, but as a passing incident to the design of the structures, it is essential that the procedure be simple and without special exceptions. The structural engineer has so many things to think about that it seems very unfortunate to load up his "tool chest" with a great many pretty but rather complicated "gadgets." Although developed in a class-room, the method of this paper was intended for use in offices.

The writer is especially pleased that so many have found the method useful in the design of actual structures. Professor Caughey and others have given estimates showing savings of from 80 to 90% of the time required by other methods; the writer knows of cases in which complete studies have required only one-twentieth of the time formerly used.

The writer is indebted to Mr. Findley for emphasizing the accuracy of the method. The writer called it a method of successive approximation; a better term which the writer has often used in this connection is "successive convergence." As Professor Van den Broek states, the word "except" scarcely has a place in the engineer's vocabulary; but the term "approximate" is one which many people misunderstand. The quest of the absolute is a beautiful thing; but he who seeks in engineering analysis a precision that cannot be ultimately translated into such units as pounds of steel and yards of concrete is misled. Structures are analyzed so that they may be designed; not for the pleasure or practice of analyzing them. As Professor Findley well states, "between the analysis of a given structure, which is essentially mathematics, and the design of a required structure, which is essentially art, lie many difficulties."

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

SUSPENSION BRIDGES UNDER THE ACTION OF LATERAL FORCES

Discussion

BY MESSRS. P. L. PRATLEY AND CHARLES A. ELLIS.

P. L. PRATLEY,[3] ESQ. (by letter).[3a]—A notable and ingenious contribution to the modern and more precise application of scientific analysis to the design of suspension bridges is presented in this paper. The authors' adaptation of the Melan exact theory regarding the distribution of load between cable and stiffening truss to actual design has become established among engineers concerned with this type of structure and their demonstration regarding the contribution of the upper chords of a stiffening truss without top laterals in resisting horizontal bending moments, while perhaps not so well known, is of similar economic value.

The desirability of some analysis regarding the "restitution" accomplished by the cable when the lateral truss deflects horizontally under wind pressure has been in the writer's mind for some years and he, therefore, appreciates highly the investigations published in this paper.

The suggested elastic distribution method, while involving considerable computation by trial and error, is intrinsically simple, and with the examples presented in the paper at hand for guidance, should be readily applicable to those cases in practice that are likely to need it. Happily perhaps, from the point of view of the designer, only spans of considerable length are to be found in this category, and as the authors show even the Detroit span (1 850 ft.) is not of sufficient magnitude to suffer from any serious quantitative inaccuracy in proportioning, if the simpler system is used in the calculations. The actual shape of the curve for z is interesting to the writer because, in studying some suspension designs in his office, subsequent to the work on the Detroit Ambassador Bridge, he assumed (as an approximate interpretation of

NOTE.—The paper by Leon S. Moisseiff, M. Am. Soc. C. E., and Frederick Lienhard, Esq., was published in March, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Cons. Engr. (Monsarrat & Pratley), Montreal, Que., Canada.

[3a] Received by the Secretary April 12, 1932.

the rather obvious truth that the rate of participation by the cable varies along the span), a triangular distribution having a maximum at the center. In the actual case subjected to analysis at the time, the proportion at mid-span was 37.5% of W_t, taken by the cable. While no analysis was made at the time to substantiate this choice and while it is evident that the maximum central figure is a function of the geometrical and physical properties of the combined truss-and-cable system, it is gratifying to note that the actual variation, as demonstrated, is somewhat of this nature. The pertinent figures in the case were: $l = 1\,500$ ft.; $h_c = 170$ ft.; $h_t = 190$ ft. to the plane of the floor; $I = 114\,000$ in.^{2} ft.^{2}; $w_t = 400$ lb.; $w_c = 100$ lb.; and $H = 9\,000$ kips. Therefore, by the authors' simpler method, $a = 3.44 \times 10^{15}$; $b = 6.04 \times 10^{15}$; and $z = 81.5$, thus showing a "restitution" factor of 20.75 per cent. For this length of span, however, it is manifest from the paper that the simpler method furnishes a satisfactorily close approximation for the truss moments. The fact that z becomes negative in Figs. 3, 7, and 8, while not expected perhaps from a first consideration, is nevertheless quite capable of physical appreciation as one visualizes the trend of actual deflections. The writer is tempted to wonder whether the peak percentage at the center could be expressed approximately for the first trial computations as a function of I, H, and l, but not having the necessary figures for all the spans treated by the authors he has not been able to pursue this inquiry to any length.

The importance of the variation in these long spans is well brought out in the graphical presentation of the moment curves, Figs. 7, 3, and 8, for the old and new Golden Gate designs and the George Washington Bridge. It is presumed that in Fig. 7 the 2 640-ft. span is treated as a pure suspension span comparable to the others, without any attempt being made to gauge the effect of the cantilever arm distortions which must occur under the assumed wind load. It might be of interest to inquire whether the truss lateral deflection is considered as taking place at the floor elevation, or elsewhere, and whether the floor system, lateral system, and truss web members are calculated as contributing to the lateral moment of inertia, in addition to the four chords?

The conclusions under "Comparative Results of Six Bridges" appear to be sound, and the economy resultant upon the use of the more exact method is decidedly worthy of notice both for its own sake and as being one further demonstration of the valuable effect of closer analysis for this type of indeterminate structure. The closing remarks of the authors on the effect of lateral stiffness are also of grave importance and serve to raise again in the writer's mind the question of "cradling" cables. One wonders whether this question too might be made the subject of some up-to-date study.

In concluding, the writer would again express his feeling that the paper is of tremendous value to those interested in this branch of bridge design, and he feels sure that the methods suggested will be accepted with gratitude by the profession and from them will develop a recognized improvement in practice and a still further economy in design.

CHARLES A. ELLIS,[4] M. AM. SOC. C. E. (by letter).[4a]—In this paper the authors have made a contribution to the theory of suspension bridges which is at once unique and valuable—unique from an academic viewpoint because it shows clearly how the cables, hangers, and stiffening trusses participate and co-operate in performing one of the duties for which the structure as a whole is designed; and valuable from a practical engineering standpoint because it gives a rational solution, in which no unwarranted liberties have been taken, to a hitherto perplexing problem. In reading the paper one must admire what amounts to genius displayed by the authors in the skilful manner whereby they have correlated and combined three simple fundamental principles most effectively in accomplishing their objective. These principles are the deflection of a simple beam, the funicular polygon, and the triangle of forces.

Since learning of the "Elastic Distribution Method" from the authors some years ago, the writer has had occasion to use it in several instances; and in doing so has discovered an algebraic solution which obviates the necessity of limiting the method to the "cut-and-try" solution described in the paper. Both the algebraic and the "cut-and-try" solutions give identical results, and the choice between them is a matter of personal preference. The algebraic solution of the authors' "elastic distribution method" will be illustrated by two examples. The first is a solution for maximum bending moments; the second, for maximum shears. In so far as possible the authors' nomenclature will be followed.

It may be well to emphasize a few points, as follows:

(1) A clear distinction should be kept in mind between the unit wind pressure acting on the trusses, which is a known constant quantity represented by w_t, and the unit wind pressures sustained by the trusses, which are unknown variable quantities represented by p_{tx} and for which the deflections, δ_{tx}, are to be determined as in a simple beam by the semi-graphical process of integration by summation. This process should commend itself to all engineers (professors and practitioners alike) because of its superiority over the classic and generally taught method of algebraic integration, which becomes very cumbersome if the moment of inertia varies or the load cannot be expressed as a continuous and simple function of one variable.

(2) A similar distinction should also be recognized between the unit wind pressure acting on the cables, which is a known constant quantity represented by w_c, and the unit wind pressures sustained by the cables, which are unknown variable quantities represented by p_{cx} and for which the deflections, δ_{cx}, are to be determined upon the conservative assumption that the cables resist the moment of the forces at any point, not by virtue of their stiffness *per se*, but rather by the product of the horizontal pull, H, and the deflection, δ_{cx}.

(3) The differences, $w_t - p_{tx} = z_x$, are the unit wind pressures transferred from the trusses to the cables by virtue of the inclination of the hangers, brought about by the difference in the deflections of the trusses and cables and represented by $\Delta\,\delta_x$.

[4] Vice-Pres., The Strauss Eng. Corporation, Chicago, Ill.

[4a] Received by the Secretary April 18, 1932.

(4) The truss deflections, δ_{tx}, are measured from the vertical plane through the longitudinal center line of the span at the truss elevation; while the cable deflections, δ_{cx}, are measured from the vertical plane through the tower tops. The horizontal distance, T, between these planes at either tower is the transverse deflection of the tower top when referred to the center line of the tower at the truss elevation. When the wind load is symmetrical, T is the same at each tower and the two planes are parallel; otherwise, T is a variable. The difference between the truss and cable deflections at any point, therefore, is,

$$\Delta\delta_x = \delta_{tx} - (\delta_{cx} + T)$$

or,

$$\delta_{tx} - \delta_{cx} = \Delta\delta_x + T \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

(5) The hanger lengths, h_x, and the vertical loads, p_{sx}, sustained by the hangers are known; hence, from the triangle of forces,

$$h_x : \Delta\delta_x :: p_{sx} : z_x$$

or,

$$\Delta\delta_x = \frac{h_x}{p_{sx}} z_x$$

whence,

$$\delta_{tx} - \delta_{cx} = \frac{h_x}{p_{sx}} z_x + T \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

The two examples which follow are taken from computations made by the writer in designing the Golden Gate suspension span. The reader will note that the general data given herewith differ slightly from those used by the authors in their Proposed Design No. 2 which is an earlier study of the same structure.

Example 1.—Maximum Bending Moments.—Let $l = 4\,200$ ft.; $h_c = 475$ ft.; dead load = 15.3 kips per ft. of bridge; live load = 4 kips per ft. of bridge; $p_{sx} = 19.3$ kips per ft. of bridge; $w_t = 1.1$ kips per ft.; $w_c = 0.2$ kips per ft.; H = horizontal cable pull per bridge for live and dead loads = 118 310 kips; $E = 29\,000$ kips per in.^{2}; and I (constant) = 1 479 110 in.^{2} ft.^{2} The span is divided into twenty equal parts, each division having the length, $\lambda = \frac{4\,200}{20} = 210$ ft. Then, $\frac{\lambda^4}{EI} = 0.0453397$; and, $\frac{\lambda^2}{H} = 0.37288$.

The expressions in Table 4 relate to the deflections of the trusses. The column numbers correspond to those used by the authors in their Table 1.

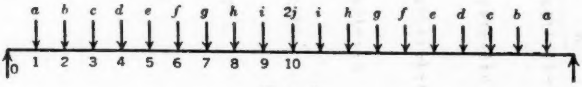

FIG. 9.

The load sustained by the trusses and for which the deflections are to be determined is of varying intensity. In Fig. 9, let $a, b, \ldots i, 2j$, represent the intensity of this load at the various division points. The intensity, $2j$, was used instead of j, in order to simplify the coefficients in the columns which follow. The

TABLE 4.—EXPRESSIONS RELATING TO THE DEFLECTIONS OF THE TRUSSES (EXAMPLE 1).

Division points (1)	p_{tx}, in kips per foot (2)	S'_x, in kips per foot (3)	M'_x, in kips per foot (4)	S_{mx}, in kips per foot (7)	M_{mx}, in kips per foot (8)
1	a	$a+b+c+d+e+f+g+h+i+j$	$a+b+c+d+e+f+g+h+i+j$	$9.5a+18b+25.5c+32d+37.5e+42f+45.5g+48h+49.5i+50j$	$9.5a+18b+25.5c+32d+37.5e+42f+45.5g+48h+49.5i+50j$
2	b	$b+c+d+e+f+g+h+i+j$	$a+2b+2c+2d+2e+2f+2g+2h+2i+2j$	$8.5a+17b+24.5c+31d+36.5e+41f+44.5g+47h+48.5i+49j$	$18a+35b+50c+63d+74e+83f+90g+95h+98i+99j$
3	c	$c+d+e+f+g+h+i+j$	$a+2b+3c+3d+3e+3f+3g+3h+3i+3j$	$7.5a+15b+22.5c+29d+34.5e+39f+42.5g+45h+46.5i+47j$	$25.5a+50b+72.5c+92d+108.5e+122f+132.5g+140h+144.5i+146j$
4	d	$d+e+f+g+h+i+j$	$a+2b+3c+4d+4e+4f+4g+4h+4i+4j$	$6.5a+13b+19.5c+26d+31.5e+36f+39.5g+42h+43.5i+44j$	$32a+63b+92c+118d+140e+158f+172g+182h+188i+190j$
5	e	$e+f+g+h+i+j$	$a+2b+3c+4d+5e+5f+5g+5h+5i+5j$	$5.5a+11b+16.5c+22d+27.5e+32f+35.5g+38h+39.5i+40j$	$37.5a+74b+108.5c+140d+167.5e+190f+207.5g+220h+227.5i+230j$
6	f	$f+g+h+i+j$	$a+2b+3c+4d+5e+6f+6g+6h+6i+6j$	$4.5a+9b+13.5c+18d+22.5e+27f+30.5g+33h+34.5i+35j$	$42a+83b+122c+158d+190e+217f+238g+253h+262i+265j$
7	g	$g+h+i+j$	$a+2b+3c+4d+5e+6f+7g+7h+7i+7j$	$3.5a+7b+10.5c+14d+17.5e+21f+24.5g+27h+28.5i+29j$	$45.5a+90b+132.5c+172d+207.5e+238f+262.5g+280h+290.5i+294j$
8	h	$h+i+j$	$a+2b+3c+4d+5e+6f+7g+8h+8i+8j$	$2.5a+5b+7.5c+10d+12.5e+15f+17.5g+20h+21.5i+22j$	$48a+95b+140c+182d+220e+253f+280g+300h+312i+316j$
9	i	$i+j$	$a+2b+3c+4d+5e+6f+7g+8h+9i+9j$	$1.5a+3b+4.5c+6d+7.5e+9f+10.5g+12h+13.5i+14j$	$49.5a+98b+144.5c+188d+227.5e+262f+290.5g+312h+325.5i+330j$
10	$2j$	j	$a+2b+3c+4d+5e+6f+7g+8h+9i+10j$	$0.5a+b+1.5c+2d+2.5e+3f+3.5g+4h+4.5i+5j$	$50a+99b+146c+190d+230e+265f+294g+316h+330i+335j$

intensity, a, at Point 1, may be taken as the average for the division length, λ, of which the point, 1, is the middle. Hence, the load over this division length may be replaced by the concentrated load, λa, acting at Point 1. Therefore, the quantities in Column (2), Table 4, and as shown in Fig. 9, when multiplied by λ, may be treated as concentrated loads at the division points. The expressions in Column (3), when multiplied by λ, represent the net reaction and shears between the division points; and those in Column (4), when multiplied by λ^2, represent the bending moments at the division points. From these data a moment diagram may be constructed, the area of which is now to be considered as a load on the structure. This load may be represented by a series of concentrated loads at the division points, each load being the product of λ and the moment ordinate for that point. The expressions in Column (4), when multiplied by λ^2, represent the moment ordinates; and when multiplied by λ^3, they represent the area-moment loads. For this loading the expressions in Column (7), when multiplied by λ^3, represent the net reaction and shears which, when divided by $E\ I$, represent the slope of the elastic curve. The expressions in Column (8), when multiplied by λ^4, represent the moments for said loading which, when divided by $E\ I$, represent the deflections, δ_{tx}, Columns corresponding to the authors' Column (5) and (6), Table 1, have been omitted since they serve only when I cannot be assumed constant.

Writing the expressions in Columns (3), (4), (7) and (8), Table 4, is little more than a procession of mechanical operations in addition and can be done very quickly; moreover, an independent check can be made by observing the following regularity in the coefficients of the terms in the expressions of Column (8). Beginning with the first term in the first row observe the same sequence in coefficients, whether reading to the right in the row or downward in the column; and the same is true when beginning with the second term in the second row, or the third term in the third row, etc. Hereafter, this phenomenon will be referred to as the "rule of coefficients".

TABLE 5.—EXPRESSIONS RELATING TO THE DEFLECTIONS OF THE CABLES (EXAMPLE 1).

Division points (1)	p_{cx}, in kips per foot (10)	S_{0x}, in kips per foot (11)	M_{0x}, in kips per foot (12)
		12.35—a—b—c—d—e—f—g—h—i—j	
1.......	1.3—a		12.25—a—b—c—d—e—f—g—h—i—j
		11.05—b—c—d—e—f—g—h—i—j	
2.......	1.3—b		23.40—a—2b—2c—2d—2e—2f—2g—2h—2i—2j
		9.75—c—d—e—f—g—h—i—j	
3.......	1.3—c		33.15—a—2b—3c—3d—3e—3f—3g—3h—3i—3j
....	....	....	....
		0.65—j	
10.......	1.3—2j		65.00—a—2b—3c—4d—5e—6f—7g—8h—9i—10j

The expressions in Column (8), Table 4, remain the same for any structure in which twenty divisions are used.

Corresponding expressions for the deflections of the cables are compiled as demonstrated in Table 5. The total unit wind pressure acting on the cables and trusses is 1.3 kips per ft., and the expressions in Column (10),

Table 5, follow directly from Column (2), Table 4, which, when multiplied by λ, represent the concentrated loads sustained by the cable. The expressions in Column (11), Table 5, when multiplied by λ, represent the net reaction and shears; and those in Column (12), when multiplied by λ^2, represent the moments which, when divided by H, represent the cable deflections, δ_{cx}, when referred to a vertical plane through the tower tops.

The expressions in Column (12) remain the same for any structure in which twenty divisions are used, except the numerical term in each expression, which is proportional to the total unit wind pressure on the structure. Note, also, that beginning with a in the first row of Column (12), the rule of coefficients applies.

The unit of measure of all terms and expressions in Tables 4 and 5 is kips per foot, or $\frac{\text{kips}}{\text{feet}}$; hence, the following units of measure:

Shear:

$$\lambda\, S'_x = \text{feet} \times \frac{\text{kips}}{\text{feet}} = \text{kips}$$

Bending moment:

$$\lambda^2 M'_x = \text{feet}^2 \times \frac{\text{kips}}{\text{feet}} = \text{kip-feet}$$

Shear for area-moment loads:

$$\lambda^3 S_{mx} = \text{feet}^3 \times \frac{\text{kips}}{\text{feet}} = \text{kip-feet}^2$$

Slope of elastic curve:

$$\frac{\lambda^3}{EI} S_{mx} = \frac{\text{feet}^3}{\frac{\text{kips}}{\text{inches}^2} \times \text{inches}^2\ \text{feet}^2} \times \frac{\text{kips}}{\text{feet}} = \frac{\text{feet}}{\text{feet}} \text{ (an abstract number)}$$

Deflection:

$$\delta_{tx} = \frac{\lambda^4}{EI} M_{mx} = \frac{\text{feet}^4}{\frac{\text{kips}}{\text{inches}^2} \times \text{inches}^2\ \text{feet}^2} \times \frac{\text{kips}}{\text{feet}} = \text{feet}$$

Shear or transverse component in cables:

$$\lambda\, S_{ox} = \text{feet} \times \frac{\text{kips}}{\text{feet}} = \text{kips}$$

Deflection:

$$\delta_{cx} = \frac{\lambda^2}{H} M_{ox} = \frac{\text{feet}^2}{\text{kips}} \times \frac{\text{kips}}{\text{feet}} = \text{feet}$$

The expressions relating to the hangers are found for the ten points, as demonstrated in Table 6. The hanger lengths are given in Column (14), from which follows Column (15), since $p_{sx} = 19.3$ kips is constant. The expressions in Column (16) follow directly from Column (2), Table 1, since $z_x = w_t - p_{cx}$. Column (117) has been added to include the tower-top deflec-

tions, $T = 1.142$ ft. Taking these into account, the fundamental equation may be written as follows:

$$\delta_{tx} - \delta_{ox} = \Delta\delta_x + T$$

or,

$$\frac{\lambda^4}{EI} M_{mx} - \frac{\lambda^2}{H} M_{ox} = \Delta\delta_x + T$$

whence,

$$0.0453397\ M_{mx} - 0.37288\ M_{ox} = \Delta\delta_x + 1.142 \ldots\ldots\ldots\ldots (14)$$

By substituting the expressions for M_{mx}, M_{ox}, and $\Delta\delta_x$ in Equation (14), ten independent simultaneous equations containing ten unknown quantities may be written and solved. These equations have been omitted in this discussion. If the equations are written consecutively, beginning with Equation 1

TABLE 6.—EXPRESSIONS RELATING TO THE DEFLECTIONS OF THE HANGERS (EXAMPLE 1).

Division points (1)	h_x, in feet (14)	$\frac{h_x}{p_{sx}}$ in feet² per kip (15)	z_x, in kips per foot (16)	$x = \frac{h_x}{p_{sx}} z_x$, in feet (17)	$x + T = x + 1.142$, in feet (117)
1..................	408.81	21.182	1.1—a	23.300—21.182a	24.442—21.182a
2..................	324.66	16.822	1.1—b	18.504—16.822b	19.646—16.822b
3..................	250.41	12.975	1.1—c	14.273—12.975c	15.415—12.975c
....	......	......	......	....	....
10..................	7.86	0.407	1.1—2j	0.448— 0.814j	1.590— 0.814j

for Point 1, with each equation transposed so that the unknown terms appear in alphabetical order beginning with a, and all numerical terms combined in the second member, a check of the coefficients may be made by the rule of coefficients. The labor involved in the solution is greatly reduced if a computing machine is used. A few hints concerning the solution may not be out of place. Begin with a and proceed alphabetically in the elimination. Note that in each equation one unknown quantity has a comparatively large coefficient, a in Equation 1, b in Equation 2, etc. This fact makes possible the choosing of one equation for the subtrahend in each elimination so that few, if any, changes in signs will result.

Several methods of solution by approximation are available to one favoring this procedure. Perhaps the best of these is assigning values to five unknown quantities in five of the equations and solving for the five remaining unknowns; for example, near the end of the truss the variable unit load sustained by the trusses differs but slightly from the load, w_t, acting on them, due to the proximity of the truss support and the distance from the trusses to the cables; and this sustained unit load decreases from the end of the trusses toward their center. Suppose that the values assigned are, $a = w_t = 1.1$; $b = 1.0$; $c = 0.9$; $d = 0.8$; and, $e = 0.7$. These values are substituted for a, b, c, d, and e in Equations 6, 7, 8, 9, and 10, the equations in which the coefficients of a, b, c, d, and e are comparatively small; and the equations are solved for f, g, h, i, and j. The values thus found are substituted in Equa-

tions 1, 2, 3, 4, and 5, and the equations solved for a, b, c, d, and e. These latter values are now taken in place of those originally assigned and the process is repeated until sufficient agreement is reached.

The results of the solution are illustrated in Column (2) of Table 7, and the remaining columns computed therefrom for a check by noting the close agreement for each division in Columns (117) and (18). The solution for maximum bending moments was the objective and, except for the check, the computations need not have been carried beyond Column (4).

The length of trusses between supports is 4 148 ft.; consequently, in finding the reaction, shears, and bending moments for the trusses, it is probably more nearly accurate to consider the length of each division as, $\lambda = \frac{4\,148}{20} = 207.4$; or $\lambda^2 = 4\,302$.

The bending moments at the division points are the quantities in Column (4), Table 7, multiplied by λ^2. The bending moments at the panel points of the truss are obtained by drawing a moment diagram and scaling the ordinates at the panel points. The shear in the end division is $S = 4.4316\lambda = 919$ kips; to which is added the load on one-half the end division to obtain the truss reaction; thus,

$$R = 919 + (1.1 \times 103.7) = 1\,033 \text{ kips}$$

The angular deflection of the trusses at the end is,

$$\alpha = \frac{\lambda^3}{E\,I} S_{mx} = \frac{91.8705\,\lambda^3}{E\,I} = 0.01911$$

For the cable, the length of each division is, $\lambda = \frac{4\,200}{20} = 210$; and the cable reaction on the tower is $R = 7.9184\,\lambda + (0.2 \times 110) = 1\,685$ kips.

Example 2.—Maximum Shears.—Ten divisions were considered ample for sufficient precision in determining the shears. The computations were first made for the loading shown in Fig. 10. The expressions for the trusses in Table 8 and for the cables in Table 9, are developed in the same manner as in Example 1. Note that, beginning with the coefficient of a in the expressions for Point 1, and reading to the left and downward, the expressions in

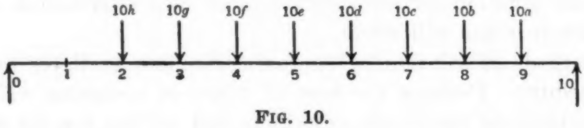

FIG. 10.

Columns (8) and (12) may be checked by the rule of coefficients. The expressions for the reactions at the top and bottom of Columns (3), (7), and (11), are determined by moments.

Because of the unsymmetrical loading, the deflections of the tower tops are unequal, and the compensating numerical term, T, which is constant in all equations for Example 1, now becomes a variable as given in Table 10. The tower-top deflections are determined from the deflection in Example 1 by assuming that the deflection of each is proportioned to the total wind reaction from cables and trusses. This assumption is, of course, only approximately

TABLE 7.—COMPUTATION OF BENDING MOMENTS (EXAMPLE 1).

Division points (1)	p_{tx}, in kips per foot (2)	S'_x, in kips per foot (3)	M'_x, in kips per foot (4)	S_{mx}, in kips per foot (7)	M_{mx}, in kips per foot (8)	$\delta_{tx} = \frac{\lambda^4}{EI} M_{mx}$, in feet (9)	$p_{cx} = 1.3 - p_{tx}$, in kips per foot (10)	S_{ox}, in kips per foot (11)	M_{ox}, in kips per foot (12)	$\delta_{cx} = \frac{\lambda^2}{H} M_{ox}$, in feet (13)	$\frac{h_x}{p_{sx}}$, in feet² per kip (15)	$z_x = 1.1 - p_{tx}$, in kips per foot (16)	$\Delta\delta_x + T = \frac{h_x}{p_{sx}} z_x + 1.142$, in feet (117)	$\delta_{tx} - \delta_{cx}$, in feet (18)
		4.4316		91.8705				7.9184						
1	1.0961		4.4316		91.8705	4.17	0.2039		7.9184	2.95	21.182	0.0039	1.22	1.22
		3.3355		87.4389				7.7145						
2	1.0297		7.7671		179.3094	8.13	0.2703		15.6329	5.83	16.822	0.0703	2.32	2.30
		2.3058		79.6718				7.4442						
3	0.9435		10.0729		258.9812	11.74	0.3565		23.0771	8.60	12.975	0.1565	3.17	3.14
......	....	....	....	....	....	....	....	....	....	....	....	....	....	....
		−0.4880		4.2664				1.1380						
10	−0.9760		8.5327		508.2434	23.04	2.2760		56.4673	21.05	0.407	2.0760	1.99	1.99

TABLE 8.—EXPRESSIONS RELATING TO THE DEFLECTIONS OF THE TRUSSES (EXAMPLE 2).

Division points (1)	p_{tx}, in kips per foot (2)	S'_x, in kips per foot (3)	M'_x, in kips per foot (4)	S_{mx}, in kips per foot (7)	M_{mx}, in kips per foot (8)
		$8h + 7g + 6f + 5e + 4d + 3c + 2b + a$		$48h + 59.5g + 64f + 62.5e + 56d + 45.5c + 32b + 16.5a$	
1	0		$8h + 7g + 6f + 5e + 4d + 3c + 2b + a$		$48h + 59.5g + 64f + 62.5e + 56d + 45.5c + 32b + 16.5a$
		$8h + 7g + 6f + 5e + 4d + 3c + 2b + a$		$40h + 52.5g + 58f + 57.5e + 52d + 42.5c + 30b + 15.5a$	
2	$10h$		$16h + 14g + 12f + 10e + 8d + 6c + 4b + 2a$		$88h + 112g + 122f + 120e + 108d + 88c + 62b + 32a$
		$-2h + 7g + 6f + 5e + 4d + 3c + 2b + a$		$24h + 38.5g + 46f + 47.5e + 44d + 36.5c + 26b + 13.5a$	
3	$10g$		$14h + 21g + 18f + 15e + 12d + 9c + 6b + 3a$		$112h + 150.5g + 168f + 167.5e + 152d + 124.5c + 88b + 45.5a$
		$-2h - 3g + 6f + 5e + 4d + 3c + 2b + a$		$10h + 17.5g + 28f + 32.5e + 32d + 27.5c + 20b + 10.5a$	
......	......	....	....	....	....
9	$10a$		$2h + 3g + 4f + 5e + 6d + 7e + 8b + 9a$		$32h + 45.5g + 56f + 62.5e + 64d + 59.5c + 48b + 28.5a$
		$-2h - 3g - 4f - 5e - 6d - 7c - 8b - 9a$		$-32h - 45.5g - 56f - 62.5e - 64d - 59.5c - 48b - 28.5a$	

correct, but is used in lieu of a more exact method simply because the wind reactions thus found are statically determinate, as if the whole structure were a simple beam. Having thus established the tower deflections, the deflection, T, at each point, is found by interpolation. Full live load was assumed

TABLE 9.—EXPRESSIONS RELATING TO THE DEFLECTIONS OF THE CABLES (EXAMPLE 2).

Division points (1)	p_{cx}, in kips per foot (10)	S_{0x}, in kips per foot (11)	M_{0x}, in kips per foot (12)
		4.68—8h—7g—6f—5e—4d—3c—2b—a	
1......	0		4.68—8h—7g—6f—5e—4d—3c—2b—a
		4.68—8h—7g—6f—5e—4d—3c—3b—a	
2......	1.3—10h		9.36—16h—14g—12f—10e—8d—6c—4b—2a
		3.38+2h—7g—6f—5e—4d—3c—2b—a	
3......	1.3—10g		12.74—14h—21g—18f—15e—12d—9c—6b—3a
		2.08+2h+3g—6f—5e—4d—3c—2b—a	
......	......	......	......
9......	1.3—10a		5.72—2h—3g—4f—5e—6d—7c—8b—9a
		—5.72+2h+3g+4f+5e+6d+7c+8b+9a	

as in Example 1: $\lambda = \dfrac{4\,200}{10} = 420$; $\dfrac{\lambda^4}{E\,I} = 0.725435$; and, $\dfrac{\lambda^2}{H} = 1.49151$.

Eight equations are required and only eight can be written because, while there are nine expressions each for M_{mx} and M_{ox}, there is no expression for $\Delta\delta_x$ for Point 1. The equations are written for Points 2 to 9, inclusive, and numbered accordingly. The equations are omitted in this discussion. If each equation is transposed so that the first member contains the unknown terms in reverse alphabetical order beginning with h, the coefficients may be checked by the rule of coefficients.

TABLE 10.—EXPRESSIONS RELATING TO THE DEFLECTIONS OF THE HANGERS (EXAMPLE 2).

Division points (1)	$\dfrac{h_x}{p_{sx}}$, in feet² per kip (15)	z_x, in kips per foot (16)	$x = \dfrac{h_x}{p_{sx}} z_x$, in feet	T, in feet	$x + T$, in feet (117)
2....................	9.640	1.1—10h	10.604— 96.40h	1.007	11.611— 96.40h
3....................	4.511	1.1—10g	4.962— 45.11g	1.015	5.977— 45.11g
4....................	1.433	1.1—10f	1.576— 14.33f	1.020	2.596— 14.33f
....	......	....	....	....	....
9....................	16.822	1.1—10a	18.504—168.22a	1.067	19.571—168.22a

Much labor may be saved if the elimination begins with a and proceeds alphabetically until the value of h is found; for by so doing the solution for the shears when Points 3 to 9, 4 to 9, and 5 to 9 are loaded, are much simplified. To illustrate, suppose that the equations are to be written when Points 3 to 9, inclusive, are loaded. In Table 8, Column (2), Point 2, make $p_{tx} = 10\,h = 0$, and the only change in all the expressions in this table is the elimination of all terms containing h. In Table 9, Column (10), Point 2, make $p_{cx} = 1.3 - 10\,h = 0$, and the only changes are in the numerical terms and the elimination of all terms containing h. The expressions for $\Delta\delta_x$ for Points 3 to 9, in Column (17), Table 10, remain unchanged; but the tower deflections, T, will

be changed. The result is that the first member of Equations 3 to 9 will be precisely the same as in the previous solution, except that the first term in each equation containing h has disappeared. All the work, therefore, in the solution will be confined to the same manipulation of the numerical terms in the second member of the equations that was followed in the previous solution.

The values of S''_x, as determined for the various loadings, are given in Table 11.

TABLE 11.—COMPUTATION OF SHEARS (EXAMPLE 2).

Division points	Points Loaded			
	S'_x, in kips per foot (2)—(9)	S'_x, in kips per foot (3)—(9)	S'_x, in kips per foot (4)—(9)	S'_x, in kips per foot (5)—(9)
0				
	+1.18778	+0.71825	+0.42800	+0.26628
1				
	+1.18778	+0.71825	+0.42800	+0.26628
2				
	+0.31675	+0.71825	+0.42800	+0.26628
3				
	−0.27500	+0.03477	+0.42800	+0.26628
4				
	−0.33906	−0.17735	+0.09006	+0.26628
5				
	+0.42156	+0.43602	+0.54931	+0.73535
6				
	+0.43896	+0.46181	+0.49258	+0.59609
7				
	−0.08558	−0.07474	−0.05226	+0.01642
8				
	−0.91430	−0.90582	−0.88507	−0.82847
9				
	−1.93890	−1.92948	−1.90657	−1.85076
10				

In Division 1–2, or its counterpart, Division 8–9, the maximum is 1.18778
In Division 2–3, or its counterpart, Division 7–8, the maximum is 0.71825
In Division 3–4, or its counterpart, Division 6–7, the maximum is 0.59609
In Division 4–5, or its counterpart, Division 5–6, the maximum is 0.73535

In computing the shear, $\lambda S''_x$, the length of the division points is, $\lambda = \frac{4\,148}{10} = 414.8$. The shear in the end division, 0–1, was obtained from the loading in Example 1.

The authors' paper sheds much light on the question of adequate widths of suspension bridges. If the trusses in Example 1 of this discussion were treated as a simple span, receiving no assistance from the cables, the deflection at mid-span would be:

$$\frac{5 \times 1.1 \times 4\,200^4}{384 \times 29\,000 \times 1\,479\,110} = 104 \text{ ft.}$$

With the assistance of the cables the deflection was found to be 23 ft.; hence this suspension system has 4.5 times the lateral stiffness of a simple span of the same dimensions. This fact should be sufficient evidence that any method—either rational or empirical—which quite properly may be used in arriving at the width of a simple truss span, is entirely foreign to any discussion of the required width of a suspension bridge.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

WIND-BRACING IN STEEL BUILDINGS

SECOND PROGRESS REPORT OF SUB-COMMITTEE NO. 31, COMMITTEE ON STEEL, OF THE STRUCTURAL DIVISION

Discussion

BY MESSRS. L. J. MENSCH, ROBINS FLEMING, RUDOLPH P. MILLER, C. M. GOODRICH, ALBERT SMITH, HUGH L. DRYDEN, L. E. GRINTER, P. L. PRATLEY, FREDERICK MARTIN WEISS, AND A. J. HAMMOND.

L. J. MENSCH,[8] M. AM. SOC. C. E. (by letter).[8a]—That wind pressures might be much higher once in 25 or 50 years than the pressures recommended by the Sub-Committee is well known. If the Sub-Committee found that buildings stood up under extremely higher pressures it was its business to try to discover the hitherto undisclosed additional strength of buildings as a whole.

The Sub-Committee confesses that amplitudes and frequency of vibrations are still in the realm of the unknown to its members. The writer has given an elementary solution of this mysterious subject,[9] in which he shows that amplitudes and frequencies caused by wind are dependent on two factors, namely, cantilever bending and shear deformation. While, for simplification, he considered only the wind-bracing bents in that paper, he will herein elaborate his ideas of the added strength offered by the building as a whole.

It is admitted that cantilever bending lengthens the windward and shortens the leeward columns of the bents and that the reactions from wind are transferred by the girders from windward to leeward. In tall buildings the spandril girders at right angles to the wind-bracing bents are often as stiff as the girders of the bents. Even where they are lighter it is likely that they are considerably stiffened by the masonry enclosures and, there-

NOTE.—The report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wind-Bracing in Steel Buildings, was presented at the meeting of the Structural Division, New York, N. Y., January 21, 1932, and published in February, 1932, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

[8] Civ. Engr. and Constr., Chicago, Ill.

[8a] Received by the Secretary February 25, 1932.

[9] "Deflections and Vibrations of Tall Buildings," *Journal*, Am. Concrete Inst., February, 1932.

fore, can transmit the reactions caused by the wind from the outside columns of the bents to the adjoining outside columns. This has the same effect as if the area of the outside columns of the bents were the sum of the areas of all the outside columns served by the wind-bracing bents. Such an effect will increase considerably the moment of inertia of the horizontal section of the wind-bracing bents (like the flanges of an I-beam) and will decrease the vertical stresses in all columns of the bents, as well as the deflections, amplitudes, and frequencies of vibrations.

The writer cannot agree that ordinary walls and partitions can stiffen bents in cantilever bending by direct compression or tension. The partitions are often only slightly in contact with the cross-girders and the poor workmanship obtained in New York City, and in other large cities, in brick-filling makes the transmission of vertical stresses for many stories very doubtful. It is not so, however, in the case of properly reinforced concrete walls as used in California which add considerably to the strength of the skeleton, not only in proportion of the area of the concrete, but also because girder and column lengths are thereby made a minimum, which increases the strength and stiffness of the members.

Shear actions are quite different from cantilever bending. It is quite clear that neither the outside walls at right angles to the bents, nor such partitions, can be of benefit to the bents in shear. The latter is often responsible in tall buildings for one-half and more of the total deflection under wind. The calculated relative shear deflections of the top of the columns in the lower stories with respect to the bottom of the columns are often as high as $\frac{1}{8}$ in., and more, under a 20-lb. wind. It is questionable whether partitions can take up such shortening and lengthening in one bay. For low wind loads, say, of a velocity of 30 to 40 miles per hour (which may occur quite often during a year and are responsible for more than 99% of all vibrations noticed in a building), the relative deflections are only a small fraction of those under a 20-ft. wind pressure. In this case the partitions in the wind-bracing bents may diminish deflections, amplitudes, and frequencies considerably, but since shear is responsible for only a part of the deflection no large reduction of amplitudes due to this help can be expected, and the writer would estimate this reduction at 10 to 25% in very tall buildings and somewhat more in buildings, say, 20 stories high.

The questions of the Sub-Committee in regard to vibrations can be answered, as follows:

(*a*) Where they are feasible, reinforced concrete walls and partitions greatly diminish deflections and vibrations; where not possible, the outside columns ought to be enclosed by rich concrete of as large a section as practicable. The New York City practice of using cinder concrete or tiles for fire-proofing the columns and girders, and omitting the concrete fire-proofing of the outside of the spandril girders, is unscientific and wasteful.

(*b*) The deflection probably increases more rapidly under high winds than under low winds because the skeleton obtains less help from partitions and walls.

(*c*) The tests[10] by R. H. Sherlock, Assoc. M. Am. Soc. C. E., on the structure of the wind shows that a so-called 50-mile wind varies in velocity from about 40 miles to about 60 miles per hour, that a rise from 40 to 50 miles often occurs in 1 sec., and that the total interval from 40 to about 60 miles occurs in less than 10 sec. These variations of wind pressure cause the vibrations of a structure and keep them going while the wind lasts. The variations in pressure amount to nearly 50% of the mean pressure. The amplitude of the vibrations at a point $\frac{5}{8}$ H above the base is given[11] by the increased wind pressure lasting for a quarter period of the free vibrations of a building divided by the force necessary to deflect the structure one unit distance at the $\frac{5}{8}$ H point. The amplitude is smaller below and larger above this point.

(*d*) The period of vibrations due to cantilever bending is given by a formula of the type, $T = 2\pi\sqrt{\frac{W\,H^3}{400\,EI}}$, in which, W is the weight of the building; H, the height, in feet; E, the modulus of elasticity per square foot; and I, the moment of inertia of a horizontal section of the wind-brac-system, in feet^4. For shear and bending a similar formula (Equation (57)) is given.

(*e*) A modern tall building differs from a simple cantilever because shear plays a large rôle.

The Sub-Committee took notice of the work of Mr. Albert Smith, and that of Professor W. M. Wilson, and Professor G. A. Maney, who undertook the rather Herculean task of computing the stresses in a 20-story frame by the theory of least work and the slope-deflection theory. The Sub-Committee also mentioned the simplifications of those so-called exact methods by Professor Clyde T. Morris and by Professor Hardy Cross[11a]. The writer claims that even these simplified methods are still too cumbersome for the practitioners; and besides, as offered, these methods give stresses and deflections which are 20 to 35% too high on account of using theoretical lengths instead of clear spans. The writer has shown[12] how to obtain more proper values in not much more time than it takes by the rule-of-thumb methods.

The Sub-Committee finds the shear deflection in the Wilson and Maney tower much larger than the cantilever deflection and does not state that both deflections have to be added to obtain the total deflection. Even where diagonal bracings are used, textbooks teach that the deformations of the web members have to be taken into consideration in trusses that are of a depth of one-fifth of the span, or deeper. The writer does not agree with the Sub-Committee that cross-bracings should be omitted in one bay when it is not possible to use them in other bays. His experience in the design of a 75-story building (which was not constructed) showed that deflections and vibrations were considerably decreased by arranging heavy reinforced concrete walls back

[10] *Paper No. 20*, Eng. Research Div., Univ. of Michigan, May, 1931.

[11] *Journal*, Am. Concrete Inst. February, 1932, p. 403.

[11a] Note typographical error in Fig. 5 of report as published in February, 1932, *Proceedings;* the distances, l_B and l_T, should be transposed.

[12] Journal, Am. Concrete Inst., February, 1932, p. 402.

of the elevators. It is true that the theory of such wind-bracing is rather complex and has not been published before, but this is no reason for discouraging the use of such bracing.

ROBINS FLEMING,[13] ESQ. (by letter).[13a]—Passing at once to the recommendations made by the Sub-Committee the writer comments upon them as follows:

Conclusions (1) and (2).—The writer is in agreement with these recommendations.

Conclusion (3).—It is not clear why the permissible stress on members subjected to wind action alone should be less than for combined action of wind and other loads, unless it is that the probability of a maximum wind load being brought upon the structure is greater than the probability of combined loads at the same time. Perhaps it may be to reduce the lateral deflection due to wind. In either case it is not clear why bolts and rivets should be excepted. However, no objection will be raised to the recommendation.

Conclusions (4) and (5).—These two recommendations will be the storm center of the entire report—provided there is a storm. The Cross method of moment distribution is far-reaching. The Appendix to the report shows how it can be applied to shallow systems of wind-bracing. Without doubt the method is more nearly exact than either the conventional cantilever or portal methods. It certainly requires only a small fraction of the work of the so-called exact methods. The question arises: To what extent will it be followed? Two or three observations will be made.

In the first place the same structural engineer is seldom called upon to design buildings as high as fifteen stories. He has probably been out of college several years. This necessitates his becoming familiar with the method. The calculation, as the report states, "must be done with care and intelligent consideration. It is naturally not a task for the office boy, but should have the best attention of an experienced engineer." Before applying the Cross method a preliminary design must be made by an approximate method to determine the relative "stiffness" of the members about each joint. Will the engineer give the necessary time to follow this up? He will not be misled by any "saving of labor" promised. To one familiar with any method and accustomed to giving undivided attention to calculation the time required is far less than that which would be taken by any one else. It is noted that three of the four members signing the report are widely known professors of engineering. It is the custom of the writer, when he sees the time stated by a professor in which an operation can be performed, to multiply the time given by three or four. In reading Rankine, when he comes across expressions such as, "it is readily seen that * * *," or "we learn from geometry that * * *," he knows that he has an evening's work ahead.

Again, are not the conventional methods, with their shortcomings, sufficiently accurate for the great majority of buildings known as skyscrapers? For the "tower" buildings of New York City and elsewhere, some of which

[13] With Am. Bridge Co., New York, N. Y.

[13a] Received by the Secretary March 7, 1932.

have a height of 500 ft., or more, the time spent in applying the Cross method is well worth while. For the great majority of 10 to 20-story buildings, in which the width at the base is more than one-fourth the height, the conventional methods, although not specially commended, give ample security.

When the students now in college, who are taught the Cross method, come upon the stage of action as designers of high buildings a revision in present practice may be expected. It is doubtful whether any widespread change, except in tower buildings, will be made before that time.

Conclusion (6).—This recommendation is excellent, but its use is somewhat limited by Conclusion (7).

Conclusion (8).—This recommendation should not be questioned.

Conclusion (9).—This recommendation is vague unless the term, "reasonable stress," is defined. In the 67-story Cities Service Building, in New York City, 950 ft. high, the owner's engineer expressed a wish that no rivets be used in tension. His wish was followed "as far as practicable." In a 36-story building recently completed, 24 000 lb. per sq. in. was used in wind connections for rivets in tension.

Conclusion (10).—It is important that this recommendation be followed. It might be well to add that provision should always be made to resist the pull from wind. A thin web-plate in a column may require strengthening. The column flange should also be thick enough to withstand the pull from rivets.

RUDOLPH P. MILLER,[14] M. AM. SOC. C. E. (by letter).[14a]—The report of the Sub-Committee refers to walls and partitions, and very properly suggests that they should not be given credit in strength calculations. No mention, however, is made of floors. When suitably designed they undoubtedly increase the stability of the structure. Consideration of this point is desirable at this time because of the recent development of lighter forms of floor construction, which may or may not provide the desirable rigidity. The steel-joist floor is an instance. Within certain limitations this is an eminently useful type of construction, but it is seriously questioned whether it is suitable when buildings reach heights at which wind forces should be taken into account.

Some engineers, notably those interested in the promotion of this new floor construction, hold that the entire resistance to distortion should be taken up in the steel frame. It would seem to be better engineering to let part of that work be done, when it can be done safely, by other elements of the structure that are necessarily a permanent part of it. Furthermore, the writer's observations at Miami, Fla., after the hurricane of September, 1926, led to the conclusion that in tall buildings there is considerable torsion due to wind. This effect was also noted by the late E. A. Stuhrman, M. Am. Soc. C. E., who, in connection with certain damage claims, made survey measurements of the horizontal distortion in a specific case.

[14] Cons. Engr., New York, N. Y.

[14a] Received by the Secretary March 16, 1932.

In the Meyer-Kiser Building, the floor construction in the rear and narrow part of the building (although it was of the same type throughout, but varying in thickness) was installed within the depth of the beams and girders, affording the maximum potential stiffness, whereas, in the front and wider section, the major part of it was above the top flanges of the beams. It was the front part of the building that failed; the lines of stress in the floor, due to torsion, were clearly marked in the floor-slabs. There were no such indications in the narrow and stiffer end of the building.

C. M. Goodrich,[15] M. Am. Soc. C. E. (by letter).[15a]—In connection with the useful and suggestive Second Progress Report of the Sub-Committee on Wind-Bracing in Steel Buildings it is believed that the method outlined therein may be modified to advantage. Noting that a large part of the wind-bending moment set against the column ends in the first line of the moment distribution solution given by the Committee (Fig. 3), is lost by cancellation, it is proposed to increase these moments by 40% in the first story, by 175% in the second story, and by 200% in each higher story given in the example. If this is done, no compensations or adjustments will be necessary. If in any given case the estimate of the proper amount of modification is in error, the amount of the correction necessary may be judged closely by noting the ratio of the amount left after distribution to the amount of moment originally placed at the column end.

These corrections being added in one lot, the moment distribution process is given a chance to do its work. Three distributions suffice for an answer, and the first addition gives a result close enough for practical purposes. No subsequent adjustments are necessary. The degree of accuracy of the result may be gauged as is usual in moment-distribution calculations.

It is believed that in many high buildings it would be satisfactory, for practical purposes, to calculate the top four, and the bottom four stories, and to interpolate for the others.

Where *X*-bracing is introduced into any story assume a percentage of, say, 95, to be carried by truss action; determine the distortion in the *X*-bracing truss; and determine the moments consequent on this distortion in the columns. If this works out at, say, 3%, then the proportion taken by the *X*-bracing is $^{95}/_{98}$ of the wind shear. A good guess will make the first proportioning the last.

The same device suggested here for tier buildings may be used for any bent, and works admirably for a Vierendeel truss.

From the writer's experience he believes that each addition in the moment-distribution process should add to zero about the joint. This puts the work on the work sheet, and assists in avoiding errors.

Albert Smith,[16] M. Am. Soc. C. E. (by letter).[16a]—The unit wind pressures on tall buildings in succeeding 100-ft. spaces above the sidewalk are given in

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[15] Chf. Engr., The Canadian Bridge Co., Ltd., Walkerville, Ont., Canada.

[15a] Received by the Secretary March 23, 1932.

[16] Cons. and Designing Structural Engr. (Smith & Brown, Engrs., Inc.), Chicago, Ill.

[16a] Received by the Secretary March 23, 1932.

Table 1. In the first column, *A* designates the Sub-Committee's proposed units; *B,* the units proposed by the writer in discussing the First Progress Report of the Sub-Committee;[17] *C,* the old New York code; *D,* the amended New York code; *E,* the old code and practice in Chicago, Ill.; and *F,* the proposed new code for Chicago.

TABLE 1.—COMPARISON OF UNIT WIND PRESSURES IN SUCCEEDING 100-FOOT SPACES ABOVE THE SIDEWALK

<table>
<tr><th rowspan="2">Codes</th><th colspan="18">100-FOOT SPACES, BEGINNING AT THE SIDEWALK</th></tr>
<tr><th>1st</th><th>2d</th><th colspan="2">3d</th><th>4th</th><th>5th</th><th>6th</th><th>7th</th><th>8th</th><th>9th</th><th>10th</th><th>11th</th><th>12th</th><th>13th</th><th>14th</th><th>15th</th><th>16th</th><th>17th</th></tr>
<tr><td>*A*......</td><td>20</td><td>20</td><td colspan="2">20</td><td>20</td><td>20</td><td>21</td><td>23</td><td>25</td><td>27</td><td>29</td><td>31</td><td>33</td><td>35</td><td>37</td><td>39</td><td>41</td><td>43</td></tr>
<tr><td>*B*......</td><td>15</td><td>20</td><td colspan="2">25</td><td>35</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td><td>40</td></tr>
<tr><td>*C*......</td><td>30</td><td>30</td><td colspan="2">30</td><td>30</td><td>30</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td></tr>
<tr><td>*D*......</td><td>0</td><td>20</td><td colspan="2">20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td><td>20</td></tr>
<tr><td>*E*......</td><td>20</td><td>20</td><td>20</td><td>30</td><td>30</td><td>30</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td></tr>
<tr><td>*F*......</td><td>15</td><td>15</td><td>15</td><td>30</td><td>30</td><td>30</td><td>30</td><td>30</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td><td>....</td></tr>
</table>

In the Second Progress Report the Sub-Committee re-affirms its confidence in the unit pressures opposite *A,* and advances the following arguments:

(1) Larger pressure units would entail increase of conservatism in design, which only test data on buildings would justify.

(2) Structures designed by such loads and even less have exhibited no distress in maximum wind storms.

(3) Such reliable data as are available as to maximum velocities and pressures, may safely be discounted, since these maxima are the effects of gusts and, in the opinion of the Sub-Committee, gusts are so small that the average pressure on the building will be much lower than the gust pressure.

(4) Buildings designed to have a moderate deflection under these unit pressures will have added security because of the conservative fiber stresses used to ensure moderate deformations.

There arguments seem far from conclusive.

Concerning Item (1), the old city codes were not framed with modern maximum heights in mind. As heights increase, the question of increasing the unit pressures on higher areas naturally arises. Instead of doing this, the Sub-Committee's units are materially less for 1 000-ft. buildings than were required by former standards for buildings of lesser height. Old standards did not take account, sufficiently, of the diminution of velocity and pressures in the lower strata, and there was no recognition of shelter. There has been a vast amount of evasion in the wind design of buildings of moderate height. A large part of it has been justified technically, but a Society code cannot be framed on the basis of special conditions of shelter. City codes might make such provision, but it is well to remember that even if a structure were erected in a cavern, it would need considerable definite strength of rectitude.

The Sub-Committee is inviting the Society to go at least part way with the New York Code Committee in a radical revision downward of wind loads. The burden of proof is on the Sub-Committee.

[17] *Civil Engineering,* May, 1931, p. 702.

Item (2) is an argument frequently used by laymen, and objected to by engineers. The inference that certain loads do not exist because structures not designed for them do not fall, should not be effective in a discussion of a code of practice. Most of the lightly braced buildings are within the built-up area. Most of the taller ones have not yet passed through a major storm. Even if, in a classification of several hundred buildings, a few are found that have minimum shelter and have passed through a major storm, it would still be impossible to state whether they did not approach nearer their ultimate strength in resistance than could safely be permitted in the performance of all buildings. The performance of the 255-ft. Daily News Tower, in Miami, during the Florida hurricane, was very impressive, but even if the plans of the structure had been available and they showed that 20 lb. per sq. ft. was the maximum load that could be carried without exceeding 24 000 lb. per sq. in. of fiber stress, designers would scarcely be justified in assuming that 20 lb. per sq. ft. of pressure would have secured adequate design in a building twice as high.

In the case of Item (3), it is natural for one who walks through small gusts at the surface of the earth to think that all gusts are small, but using this inference as a basis for units of a design code to be applied to buildings of any height in any location is unjustifiable optimism. If each city block had a large tower, and all these towers were of the same height, the Sub-Committee's statement that, "the uniformly high velocity over a large frontage that is associated with a freely moving gradient wind is rendered impossible below the tops of the tallest structures present," would be quite true, except for those towers on the edge of the city. In the discussion of the First Progress Report of the Sub-Committee there was a fine view of New York towers.[18] It is not clear how any engineer can look at that view and feel that the towers shown shelter each other. Below the 20-story level the churning caused by the great number of obstacles makes the average velocity of the wind much less than at the tops of the towers, but it is inconceivable that the structure of the wind on an 800-ft. tower will be appreciably affected by the presence of another tower, 1 000 ft. to windward.

Concerning Item (4), if the stability factor and the safety factor for connections were as large as the safety factor in columns, beams, and wind struts, the large factor in the latter, adopted for stiffness, would give additional security. This is not the case. Even if it were, this would be an argument rather for acquiescence in than agreement with the Sub-Committee's report. It sounds as if the Sub-Committee had stated that "even if you get larger pressures, they will do no damage."

To find out, roughly, the difference between the unit loads, *A* and *B,* in Table 1, the writer has computed by Spurr's method the stresses in a tower of one hundred 10-ft. stories, with bents of four 20-ft. bays, and each bent carrying 20 ft. of wind exposure.

At the seventieth story from the top the wall column carrying *A* load stresses and designed by the Sub-Committee's combined compressive fiber

[18] *Civil Engineering,* May, 1931, p. 703.

stress, would have a fiber stress of 19 500 lb. per sq. in. Stresses from the *B* loading would cause the fiber stress in this column to be about 23 000 lb. per sq. in. Since the slenderness ratio of this column is about 11, the writer regards the column as adequate for the larger loads.

The moment in the beam connecting to this wall column, referred to the center line of the column, is about 330 000 ft-lb. from the *A* loading and about 530 000 ft-lb. from the *B* loading. Designing by the Sub-Committee's combined fiber stress clause, this beam would have to be designed for a fiber stress little greater than the static load fiber stress, since 330 000 ft-lb. is many times the static load moment. In a beam thus designed, the combined fiber stress from the *B* loading would probably not exceed 27 000 lb. per sq. in.

If either trussing or knee-braces were used in this case, designed with a static load fiber stress for Loading *A*, the fiber stresses from Loading *B* would be 60% greater. To keep the fiber stress in knee-braces within 150% of the static load permissible would require a little more steel.

The only important necessary cost difference between the two loadings would appear in the connections. The Sub-Committee, while placing a low limit on combined fiber stress in bracing members, with a view not only to stiffness, but also to security, allows the connecting rivets to be stressed to 18 000 lb. per sq. in. This is about as high a stress as a rivet in a well-designed lug connection should be expected to carry. There is no margin in this case for greater wind load. At the thirtieth floor in a 100-story building (designing for the same rivet stress) the *B* loading would require about 26% more rivets, and 60% more bracket steel than the *A* loading. For trusses or knee-braces the relations for rivets, gussets, and lugs would be about the same, although the total difference in cost would be much less.

In regard to stability the Sub-Committee's specification leaves little margin for additional wind load. A building 90 stories high, if framed with bents having four 20-ft. bays, would require no anchorage for the *A* loading. By the *B* loading anchorage would be required for a 60-story building.

If the braced bents had three 20-ft. bays, it would be built to sixty stories without requiring anchorage by the *A* loading. If the *B* loading were used a 40-story building would require anchorage. If the walls were lighter than the writer has assumed, or the braced bent carried more width of wind exposure than of dead load, these limiting heights would be reduced. The Sub-Committee's margin of 50% of the stresses from these very low wind loads is diminished by probable error in calculation, and by the uplifting effect of bending moment in the column.

If the combined fiber stress in rivets were reduced to 13 500 lb. per sq. in. and the stability clause required anchorage when the wind tension exceeded 45% of the dead load, the code recommended by the Sub-Committee would secure safety in careful designs. It would not be a good code to set up, however. Designing for low fiber stresses tends to make a designer less careful of his stresses, and where the stresses are so indeterminate as wind stresses in buildings, if he forgets that his wind loads are low, or does not know that they are low, he is likely to make too little allowance for error.

The Cross method of stress calculation, as exhibited by the Sub-Committee, seems very satisfactory for a bent when its share of the total load can be determined. The Sub-Committee should note that even for bents of the same length and column spacing, it is very difficult to assign the true load to the bent, and that to determine accurately what load should be assigned to bents of an L-shaped building is impossible.

Any code statement of methods of computation should include a statement of the method described by Mr. H. V. Spurr.[19] This method can only be applied to towers in which bracing bents have their neutral axes in the same planes, and in which interior bays can be made much stiffer than the exterior bays. For such structures, except for limits of design accuracy and for unequal inelastic deformation, due to slip of riveted joints, Mr. Spurr's method secures perfect action in resisting code wind loads.

In connection with setting a limit to maximum combined fiber stress in columns, the Sub-Committee should propose a proper tolerance for column milling, both for angle and for deviations from a plane surface. It may be that setting a limit to the amount of movement of the top of the free-standing column to permit riveting of beams would be the best way to frame a specification. It is clear that the ultimate strength of building columns is greatly affected by the accuracy of the fitting in the field.

HUGH L. DRYDEN,[20] ESQ. (by letter).[20a]—Wind-tunnel measurements have been made on a model of the Empire State Building, New York City, to obtain data for comparison with the results of measurements on the full-scale structure to be made by a committee of the American Institute of Steel Construction. The measurements show the distribution of wind pressure around the building at three elevations for several wind directions. The wind loads observed correspond to values of 35 to 40 lb. per sq. ft. at a true wind speed of 100 miles per hour. It is hoped that when the full-scale data are available, a convincing demonstration may be given of the validity of the results of model tests applied to wind-load determinations. In the present state of the art, the conventional design load bears no close relation to the actual wind load, and safety against wind loads which are actually much higher than the conventional design load is secured by the use of conservative values of the permissible stress, and by the neglect of the strengthening effect of walls and partitions.

Some engineers have expressed a lack of confidence in the application of the results of model tests to natural wind conditions because of the influence of gusts. Attention is directed to the experimental study[21] of gusts by R. H. Sherlock, Assoc. M. Am. Soc. C. E., and Professor M. P. Stout, at the University of Michigan. In a gusty wind, it is true that, on the average, high velocities do not occur simultaneously over large areas; but it is also true that, occasionally, they do. Furthermore, at a speed of 100 miles per hour,

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[19] "Wind Bracing", McGraw-Hill, N. Y., 1930.

[20] Chf., Aerodynamical Physics Section, Bureau of Standards, Washington, D. C.

[20a] Received by the Secretary March 26, 1932.

[21] *Bulletin*, National Elec. Light Assoc., January, 1931, and January, 1932.

a gust lasting 5 sec. must extend over a distance of about 750 ft., and, under such conditions, the flow around a building and the forces on it are not essentially different from those in a uniform flow at the same speed as that in the gust.

Advance in the knowledge of wind pressure will come, not by passive waiting for the occurrence of failures of buildings in wind storms, but by aggressive study of all aspects of the subject. The time is perhaps not yet ripe for the abandonment of the traditional conventional assumptions, which have been found by experience to be conservative, but every engineer should recognize that the experimental study of wind pressure will lead ultimately to a more rational basis for the design of structures to withstand high winds.

L. E. GRINTER,[22] ASSOC. M. AM. SOC. C. E. (by letter).[22a]—It seems to the writer that the report of the Sub-Committee on Wind Bracing in Steel Buildings reflects in large measure the common-sense viewpoint. The recommendation in regard to a prescribed wind force can be justified in that manner. A wind force of 20 lb. per sq. ft. has become almost standardized for buildings of ordinary height. Buildings designed for this wind pressure have repeatedly shown satisfactory resistance. In the undisturbed zone above the 500-ft. level, considerably higher wind velocities than those encountered at low levels seem possible. The Sub-Committee's recommendation of 30 lb. per sq. ft. at the 1 000-ft. level is as great an increase over the allowance of 20 lb. per sq. ft. below the 500-ft. level as common sense would seem to justify. If a pressure greater than 30 lb. per sq. ft. is needed at the 1 000-ft. level, the standard allowance of 20 lb. per sq. ft. below the 500-ft. level has evidently been inadequate, and this conclusion is not justified by past experience.

Many engineers seem to take the results of measured wind velocities too seriously. In the first place the maximum velocities measured undoubtedly represent gusts which would be unlikely to cover the entire face of a tower. Then there is the unquestioned resistance of partitions which is neglected when designing the frame. Finally, one must consider the limiting deflection from the viewpoint of comfort to the occupants of the building. The stipulation of a wind force to be used in design must be made with all these factors, as well as with the allowable working stresses, in mind and, therefore, its connection with measured wind velocities is not as direct as some authors would lead one to believe. In the writer's opinion it is only desirable that the variation of wind force stipulated at different levels should conform roughly with the variation in measured pressures. The actual amount of the prescribed wind force is better reached through the use of judgment and experience.

The Sub-Committee's recommendation in regard to the working stress for the design of members which carry dead load, live load, and wind stress is of importance. In the interest of obtaining a uniform reserve of strength

[22] Prof. of Structural Eng., Agri. and Mech. Coll. of Texas, College Station, Tex.

[22a] Received by the Secretary March 28, 1932.

in all members for resisting high wind loads, the Sub-Committee suggests that the working stress be allowed to vary with the ratio of wind stress to total stress. The suggestion is made that the working stress be increased $33\frac{1}{3}\%$ when the wind stress amounts to 25% of the total stress, and that the allowable increase be progressively reduced to zero as the ratio of wind stress to total stress approaches unity. The Sub-Committee makes no recommendation as to the curve of variation between these extremes. Unquestionably there is a practical advantage to be obtained from the use of a specification of this type; however, the writer feels that designers are too accustomed to the use of a basic working stress to take kindly to this suggestion. Instead, he would advise fixing the working stress and specifying that only the wind stress in excess of $33\frac{1}{3}\%$ of the dead load and live load stress be considered in design. Such a specification will accomplish the result desired by the Sub-Committee without the confusion resulting from a variable working stress. Fig. 7 shows the actual variation in working stress when wind stress to the amount of $33\frac{1}{3}\%$ of the dead load and live load stress, is neglected.

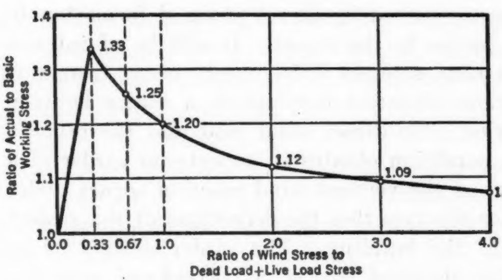

FIG. 7.—EFFECT OF DISCARDING WIND STRESS UP TO $33\frac{1}{3}\%$ OF DEAD LOAD + LIVE LOAD STRESS.

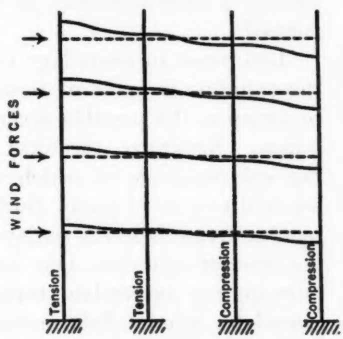

FIG. 8.—EFFECT OF COLUMN DEFORMATION.

The Sub-Committee recommends that an extension of the Cross method of balancing fixed-end moments be used in the calculation of wind moments in tall building frames. The Sub-Committee is to be congratulated for recognizing the value of this development in structural analysis so soon after its publication. There seems to be little excuse for the continued use of roughly approximate methods of analysis in this field. However, it is desirable to point out that there are available several possible extensions of the Cross method of balancing fixed-end moments, but one of which is discussed in this report. Still other variations will undoubtedly be mentioned later, and the designer should realize that there are no particular advantages in adhering exactly to the form prescribed by the Sub-Committee. It is also true that some engineers will prefer other semi-exact methods, such as the approximate slope-deflection method. If an engineer has already made himself familiar with one satisfactory method of analysis, it is not necessary that he extend himself to learn another for which he may have only an occasional use.

In an Appendix to the Sub-Committee's report is offered an example of the computation of wind moments in a building frame. The analysis is unusual in that the process of successive corrections through balancing moments and shears is stopped considerably before convergence is reached, and the final moments in the columns are obtained by a semi-graphical process based upon equal departures. The writer does not object to this procedure, but has found any such device unnecessary in his own work. The example in the Sub-Committee's report was worked out with four-place fixed-end moments. This fact suggests the tedium which probably inspired the graphical process and also explains the need found for a 20-in. slide-rule. The writer ordinarily has used three-place fixed-end moments and has found a standard 8-in. slide-rule satisfactory. The labor is greatly reduced by starting with only three significant figures. The error involved in dropping the fourth significant figure usually is of no importance whatever. In fact, two-place moments can be used successfully in analyzing simple frames. By use of this simplification, one will find it an easy matter to keep within the estimated time allowance of 10 min. per member without using the graphical method.

Reference to secondary moments caused by direct stress deformation in the columns is made at several places in the report. It will be of interest to estimate the possible error in wind stresses which might occur from this source. Naturally, the most serious situation develops in a girder between two columns, one of which carries little direct wind load and the other a heavy direct wind load. Such a condition obtains in an exterior girder of a regular frame where a major part of the vertical wind reaction occurs under the exterior columns. One further observes that the deflections of the girders increase for successive stories up the building. The girder deflections as caused by column deformation are sketched in Fig. 8. Fixed-end moments in a girder are dependent upon the relative deflections of the two ends of the girder and also upon its stiffness. It follows that large fixed-end moments might occur either in a very deep girder near the bottom of the building or in a girder of ordinary depth near the top of the building. The possible combination of a deep girder located in an upper story of the building would be particularly serious. It might be desirable to connect such a girder to the columns with clip angles only so that its moment resistance would be small.

The Sub-Committee's recommendation limits the direct column stress caused by wind to two-thirds of the dead load compression. Assuming that the dead load stress is twice the live load stress and that the direct wind stress is two-thirds of the dead load stress, the total direct stress without considering wind flexure being 15 000 lb. per sq. in., the direct wind stress is found to be limited to less than 5 000 lb. per sq. in. It will further be assumed that the average unit wind stress over the entire length of the column of a 300-ft. tower will not exceed one-half this value, or 2 500 lb. per sq. in. Commonly, the first interior column will be stressed by wind to somewhat less than one-half the direct wind stress found in the exterior column. If the average stress in the first interior column is taken at 1 000

lb. per sq. in., the differential in stress is 1 500 lb. per sq. in. For a 300-ft. tower the relative change in length between the exterior column and the first interior column will then be 0.18 in. A girder at the top of the building might reasonably be 20 ft. long and 20 in. deep. Such a girder would be stressed to 5 600 lb. per sq. in. by the fixed-end moments. This girder stress will be reduced considerably when moments are balanced, but it will still be rather large to neglect. For higher towers, more serious stresses from the effect of column deformation are to be expected.

The writer's suggestion is simply that a rough computation of this type be made after the first analysis has been completed, and, if the secondary moments could possibly become serious, that they be calculated and used in the final design. It is of interest in this connection that one analysis will seldom be sufficient in any case. It is necessary that the sizes of the members be known before an analysis can be made. Therefore, the first analysis, which is based upon preliminary member sizes, will need to be revised at least in part when certain members are increased or decreased in size. If a complete second analysis is to be made, it is a simple matter to introduce fixed-end moments into both girders and columns and thus obtain final moments which include the effect of column deformation. One cannot include this effect in the first analysis since the direct stresses in the columns are as yet unknown. A preliminary estimate of the direct stresses in the columns might be satisfactory for a highly regular frame, but would be worse than useless for an irregular structure.

The Sub-Committee is correct in pointing out that shallow bracing systems used in parallel with diagonally braced systems are likely to be of little use because of the greater deflections which may be allowable in the system with shallow bracing. An investigation which the writer made several years ago when studying open web trusses showed that the elastic deflections of such structures are at least equal to the deflections of triangulated trusses and that there are equally large deflections caused by slip in the rivets of the joints. However, it should be noted that the members of shallow-braced building frames are not stressed to their normal working stresses by wind alone, while wind-bracing of the diagonal type may be stressed to its full allowable stress by wind. This fact combined with the possible elimination of joint slip by the use of welded construction will be found to place the two systems upon nearly identical bases in so far as lateral deflection is concerned.

For instance, if the lateral deflection of the Wilson-Maney bent be computed at the top of the fourth story for a wind pressure of 20 lb. per sq. ft. and with bents spaced at 20-ft. centers, this deflection will be found to be 0.8 in. On the other hand, if diagonal bracing is placed in the outside bays and is stressed to 18 000 lb. per sq. in. by wind alone, the corresponding deflection is 1.0 in., a larger value than was obtained for the frame with shallow bracing. Both these figures represent web deflection only and, therefore, are roughly comparable. It follows that diagonally braced frames for this structure might be used in parallel with shallow-braced frames of welded construction without serious objection. The correct procedure in normal

cases is to determine the relative deflections of parallel frames and to distribute the wind force to each frame in proportion to its resistance to deflection. A rigid floor system is assumed to exist, and, if rivet slip is considered of importance, its effect upon deflections must be estimated unless all bents are of similar construction. The Sub-Committee's statement that "it is, in general, impractical to combine types of bracing of widely different essential rigidities in the same story of a bent," is an excellent safeguard, but it should not be interpreted to mean that diagonal bracing and shallow bracing can never be combined in this manner.

P. L. PRATLEY,[23] ESQ. (by letter).[23a]—In looking over the Second Progress Report of the Sub-Committee on Wind Bracing the writer finds much of interest, especially under Items (1) and (5). The variety of individual opinion is not only natural to the question, but should be regarded as a sign of healthy concern, lending vigor to the contention that only very broad principles can be laid down as being of universal application and that proper consideration must always be directed to the particular local problem.

The text of the report submitted by the Sub-Committee suggests very forcibly that the Committee is quite conscious of this fact, as both in its statement regarding "collateral stipulations" and in its expressed readiness to modify the present recommendations upon receipt of fuller information and more satisfactory evidence, it takes what, to the writer, seems the highly commendable attitude of not specifying too much. To cramp the initiative of the designing engineer or fetter his judgment by laying down stipulations and regulations from which he is not expected to depart, would be a regrettable procedure. It is evident that, while the first recommendations of the Sub-Committee were on "middle ground," the profession will continue to encounter many cases where "middle ground" must be abandoned in practice, as the problem of bracing a high building against wind must be of necessity an individual problem for which no general set of recommendations or specifications can fully provide. The recommendations of such a Committee, therefore, should draw attention to the basic principles involved, should refer possibly to the general lines along which conditions may diverge, and should advise broadly as to what considerations should be accorded to certain definite features of construction which are likely to occur with sufficient frequency to warrant such special mention and which are intrinsically likely to have some important effect on the design. Generally, the Sub-Committee does seem to be following this procedure and the writer trusts that it will not be lured away from it into "specifications."

Responsible engineers should not only be allowed, but should be encouraged, to use their own judgment in applying the fundamental principles to the actual conditions and problems which they are called upon to meet and in association with the materials available for their use. It should be their function and duty to appraise these conditions and to devise means of interpreting them in terms of structural provisions. Mandatory specifica-

[23] Cons. Engr. (Monsarrat & Pratley), Montreal, Que., Canada.

[23a] Received by the Secretary March 31, 1932.

tions, while wholesome and proper in their own sphere where they deal with the common and typical structure and where they constitute the only means for the protection of the public, only become irksome and limiting when applied to an unusual structure or imposed upon the experienced engineer responsible for its design. On the other hand, a series of memoranda such as the Sub-Committee seems now to be preparing, wherein the matters to be treated, the alternative methods of treatment, the considered opinions of professional colleagues, and available experimental data, etc., are assembled or summarized, acts rather as a stimulant, supplementing the memory and suggesting further lines of inquiry.

The writer, therefore, believes that the Sub-Committee is following the right lines when directing attention to such considerations as stability, permissible stresses on the material, the contribution of walls and partitions to stiffness, and the quantitative value of deflection. Under the heading of "Limiting Stress in Members," he agrees with the suggestion in regard to stability, that wind stress should not be allowed to overcome the dead load compression in building columns and agrees that the two-thirds limit is probably a wise precaution. This particular paragraph, however, should be rounded out with a reminder that the foundation itself must be made adequate for any uplift that may be imposed upon it.

The re-worded recommendation, with one exception, is in perfect accord with the writer's ideas and practice, and a graphical representation of this stress limit is presented in Fig. 9 with the idea that it might possibly be

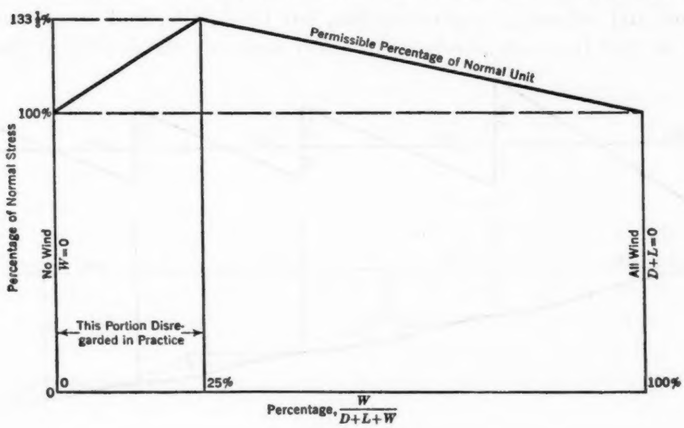

FIG. 9.

illuminating. The exception referred to is that of rivets and bolts. The writer is not as yet able to see that these should be differentiated in any way from the remainder of the structure. The theory of allowing a high working unit for a combination of wind, live load, and dead load stresses appears to arise from two grounds: First, the bridge practice of assuming that the full wind load and the full live load are not likely to occur together; and, second, that in any structure in which this combination might occur, it only persists for a very brief time and at infrequent intervals. The intention in bridge

practice, as the writer interprets it, is not so much to permit a higher unit to occur regularly with the assumed wind load, but merely to express in perhaps an awkward fashion that the actual sum of co-existing stresses does not exceed the normal permissible unit. In transferring this practice to building design engineers are perhaps a little illogical and the suggestion arises that in certain localities the wind load may be a more or less permanent condition and the liberty permitted under the recommended clause should not be taken so freely.

The limiting of vibration is an obscure problem from the quantitative point of view. No high building frame is simple enough to permit of calculations being made for its amplitude and frequency of vibration. Upper and lower limits might possibly be computed approximately from corresponding assumptions, but normally they would be far apart and the actual value would seem to be a matter for measurement after construction. To increase the moment of inertia would manifestly operate toward reduction of amplitude, other things being equal, and, therefore, any system of bracing that would tend to tie the various columns of one bent together and give the bent as a whole a moment of inertia sensibly greater than the sum of the moments of inertia of the individual columns, would serve this end.

Similarly, the more intimate the bond between the steel frame and the filling (walls, floors, etc.), the stiffer the structure must become. The relative stiffness of sheet-metal filling, riveted, bolted, or welded to the frame, and of concrete construction definitely bonded by wrapped mesh or other means to the beams and columns, is questionable, but brick, tile, and stone veneer can scarcely be tied in so effectively as to affect seriously the inertia of the frame.

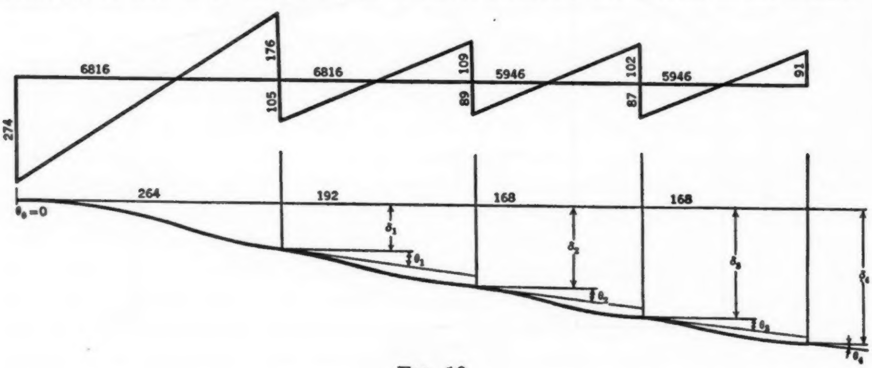

FIG. 10.

The writer cannot answer Questions (*b*), (*c*), (*d*), and (*e*) beyond expressing his "feeling" about them. The deflection as an effect must bear a relation to the wind force which is its cause, and doubtless if the wind force could be correctly appraised it would be found that the relation is linear or direct. The effect of wind impact is certain to be abstruse. Dead weight has to be moved if deflection occurs and energy, therefore, has to be expended in moving it. While the lateral force of the wind provides the energy, the transformations which this energy suffers are not so simple, and it is difficult

to understand how any definite answer can be given to Question (*d*). Regarding Question (*e*), it would appear that the modern tall building, while cantilever in its nature, is far removed from the simple uniformly stiff fixed-ended column, and the vibration is correspondingly complex.

In reading the Appendix the writer has not yet been able to discover any particular advantage, either in clarity or time, in the suggested method of computing story deflection over that which consists in treating the column according to the common elastic line theory as a cantilever fixed at the bottom end and subjected to known moments (see Fig. 10). For example:

$$E\,I\,y'' = m = 274 - 1.0708\,x$$
$$E\,I\,y' = 274\,x - 0.854\,x^2$$
$$E\,I\,y = 137\,x^2 - 0.285\,x^3$$

and similar expressions may be written for each story, from which,

$$\delta_1 = 0.0210$$
$$\delta_2 = 0.0210 + 0.0121 + 0.00294 = 0.0361$$
$$\delta_3 = 0.0361 + 0.01032 + 0.00138 = 0.0484$$
$$\delta_4 = 0.0484 + 0.00925 + 0.00217 = 0.0598$$

and,

$$E\,\theta_1 = 1\,890$$
$$E\,\theta_2 = 1\,890 - 44 = 1\,846$$
$$E\,\theta_3 = 1\,846 - 194 = 1\,652$$
$$E\,\theta_4 = 1\,652 - 54 = 1\,598$$

This solution involves the use of a 10-in. slide-rule and one sheet of computations and checks the values in Fig. 6.

While the foregoing comments offer nothing new, they are submitted as an expression of the writer's appreciation of the Sub-Committee's general attitude as reflected in this Second Progress Report.

FREDERICK MARTIN WEISS,[24] JUN. AM. SOC. C. E. (by letter).[24a]—In proposing the Cross method as the ideal mathematical analysis of wind stresses in buildings of moderate height, the Sub-Committee, in this report, repeatedly informs the reader that the twofold aim of sufficient ultimate accuracy and practical applicability with particular reference to time required was uppermost throughout the study. To discuss these two considerations justly requires that the Wilson and Maney 20-story bent, analyzed by the Sub-Committee, be regarded first as an "unclothed" steel frame, and, second, as a wind bent in an actual building.

Imagining the Wilson and Maney bent as a bare steel frame with rigid connections and subjected to lateral loads known with reasonable certainty, the Sub-Committee displayed sound judgment in advocating the Cross method for solving such a highly indeterminate structure. The two great advantages resulting from the use of this method are a tremendous saving in time and labor over that required by other "exact" methods, and any desired degree of accuracy may be attained simply by increasing the number of cycles of

[24] Princeton, N. J.

[24a] Received by the Secretary April 8, 1932.

operations. These advantages of the Cross method fully justify the viewpoint of the Sub-Committee.

Assuming that the wind loads applied to the frame are known with certainty, the results obtained after three cycles of operations appear to be sufficiently close to the correct values as to warrant no further refinement. However, the Sub-Committee evidently does not consider such results satisfactory from the standpoint of accuracy, because it uses the "semi-graphical process" to obtain the final values. The instructions that accompany Fig. 4 state that successive chords of equal departures should be drawn "making the angle of each with the preceding chord slightly greater than the last angle." Although such instructions seem quite indefinite, they are perhaps as explicit as can be given. The correct interpretation of "slightly greater" can only be determined after reproducing Fig. 4 to a large scale. The "semi-graphical process" is not based upon any correct mathematical relation, but is purely a statistical process by which hooks are added to the curves plotted from the Cross approximations. Like all statistical processes it involves the element of probable error. This is displayed in the case of the top section of Column A of the first story where the value of the moment arrived at by the "semi-graphical process" (14.72) is no nearer to the correct value (15.02) than the value as given by the third Cross approximation (15.39). Obviously different plottings of the same curve by the "semi-graphical process" would produce a different result in each case, but the important point is that all would be within the required limit of accuracy.

In respect to the time required to apply the Cross method to the Wilson and Maney bent, the writer finds that an average estimate of 10 min. per member is about correct. This speed can easily be attained after one is thoroughly familiar with the process and is not an unreasonable expenditure of time considering the resulting accuracy. In applying the Cross method one is likely to find "short-cuts" which have not been suggested by the Sub-Committee in the "Recommended Procedure in Moment Calculations for Shallow Bracing Systems" of the Appendix of the report. For example, Fig. 3 shows a part of the 20-story bent with the stiffness ratios written above each member and the sum of the stiffness ratios of all the members intersecting at the joint recorded in a circle at the joint. Every time the moments at a joint are to be balanced it is necessary to multiply the unbalanced moment by each of the quotients resulting from dividing the stiffness ratio of each member intersecting at the joint by the sum of all the stiffness ratios written in the circle. A far better plan is to evaluate a "distribution factor" for each end of a member before any of the steps of the first cycle of operations is performed. Such a factor is merely the quotient of the stiffness ratio of the member and the sum of all the stiffness ratios of the members radiating from a joint. To distribute an unbalanced moment at any joint it is simply necessary to multiply by the "distribution factor" to obtain the proportion that is to be allotted to a member intersecting at the joint. Similar factors for each of the columns may also be written to facilitate the carrying out of Step 4 of the Sub-Committee's recommended procedure. Such factors eliminate much repetiton of slide-rule computations.

Turning now from a purely academic consideration to that of regarding the Wilson and Maney bent as a part of the steel frame of an actual building, the writer does not believe that present knowledge of loading conditions and detailing connections would justify the use of such an exact method as that of Professor Cross. The Cross method, as a mathematical tool, may be likened to a very finely graduated pair of callipers capable of supplying any degree of accuracy desired. To use such a precision instrument to measure the diameter of a tree stump would be ridiculous. Yet this is analogous to applying the Cross method to a building frame subjected to loads known with no certainty whatever. The Sub-Committee realizes the approximate character of the wind load by stating under "(2) A Method of Analysis of Shallow Bracing Systems" that the wind assumption is a matter of much uncertainty. Whereas the early approximate methods of wind stress analysis lead to results which may be quite erroneous both in magnitude and sign, it now appears that the other extreme is being advocated, forgetting that no final result can be more accurate than the original data from which it is computed. One of the most common sense statements ever written in a discussion of wind stresses in buildings is that of David A. Molitor, M. Am. Soc. C. E., who stated:[25]

"The numerous assumptions necessary to bring the problem within the realm of possible solution are so grossly approximate that any attempted refinement must be excluded on the grounds of adding merely a fictitious accuracy to something which is nevertheless unknowable."

The following are some of the factors which should be known with greater certainty, than at present, before an "exact" method may really produce correct results:

1.—The variation of wind pressure over a vertical strip must be correctly established. At present, there are so many theories based upon uniform, triangular, trapezoidal, and other forms of loading that the acceptance of any one of them would make the results obtained by even the most accurate mathematical analysis quite erroneous, if the loading theory were not a correct one.

2.—The variation of wind pressure over a horizontal strip must likewise be determined correctly. Recent experiments indicate that the common practice of allotting to the edges of a building wall the same unit pressure as to the center, is not in accordance with measured pressures.

3.—If the wind shear in any story is to be distributed among the bents, so as to produce equal deflections, the stiffening effect of walls and partitions, in those bents which contain them, must be given due consideration. This calls for experimental determination of deflections of composite masonry and steel bents.

4.—Types of connections must be developed and adopted, which will actually create rigid joints.

5.—Although the stiffness ratios of the columns are little affected by the use of brick or tile fire-proofing, the stiffness ratios of the girders with their concrete fire-proofing and floor-slabs are considerably different from those of

[25] *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 189.

bare steel beams. In computing the stiffness ratios of the girders consideration must be given to the moment of inertia of not only the steel but also of the concrete fire-proofing and a tributary part of the floor-slab.

6.—The girders of a bent must be designed as partly restrained beams under vertical loads and not as simply supported beams. Unless this is done, the end connections designed for wind moment alone will be greatly overstressed by the negative moment resulting from live and dead loads.

7.—In designing wind-bracing connections more theory, substantiated by tests, must be available than at present.

Although the writer realizes that the Sub-Committee chose the Wilson and Maney bent for test purposes because it had already been solved by the slope-deflection method and not because it was supposed to be representative of an actual building frame, nevertheless, the time and labor involved in analyzing this 20-story bent is only a small fraction of that required for an actual building demanding thorough and careful wind-stress analysis. If the same frame had been unsymmetrical, the number of members requiring consideration would have immediately increased from 80 to 140. In an actual building there might be several wind bents which would have to be analyzed. For each of these bents the panel wind loads, the wind shears in each story, and the stiffness ratios of each of the members would have to be computed. These additional tasks should be included in the time estimate, since the Sub-Committee was not required to compute them, as they are listed by Professors Wilson and Maney.[28]

In one of New York's tallest skyscrapers—One Wall Street Building—there are, in all, 16 unsymmetrical wind bents, no two of which are alike. A typical bent selected through the central portion of the tower consists of 27 stories, 5 bays wide, surmounted by 26 stories, 3 bays wide, making a total of 473 members in the bent. Of the 16 bents, 9 extend to the top of the tower, and they each contain at least the number of members that is found in the typical bent described. One can thus readily see that the analysis of such a building by an "exact" method would be a colossal, if not impracticable, undertaking.

The writer also attempted to use the Cross method, as proposed in the Appendix, in the analysis of the basement, two sub-basements, and the first four stories of the aforementioned bent. Altogether there were 77 members, making the magnitude of the task correspond to the Sub-Committee's analysis of the Wilson and Maney bent. After carrying out three cycles of operations on all the members, the results were so far from satisfactory that it was estimated that probably ten or even twelve approximations would be required to approach the accuracy attained by the Sub-Committee in its three approximations. This was due to the fact that the lower story columns of an actual skyscraper are very stiff as compared with the girders framing into them. Hence, upon making a moment distribution at any joint in the stories, very little is distributed to the girders. This, in turn, requires that a great many approximations must be made to accumulate sufficient moment in the girders to counteract the moment in the columns.

[28] *Bulletin No. 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

A. J. HAMMOND,[27] M. AM. SOC. C. E. (by letter).[27a]—In connection with the wind loads prescribed by the Sub-Committee, the writer's attention has been called to tests by the Bureau of Standards on wind pressures acting through a wind tunnel.

The best way to determine whether the stresses produced by air currents through a wind-tunnel model simulates correctly the actual winds of a hurricane, cyclone, or tornado, is to select a good high-powered hurricane and have a personal experience in it.

The writer had such an experience with the Cuban hurricane of 1926, which registered more than 100 miles per hour. Awakened about 2:00 A. M., by the extraordinary whistling of the wind, he stood at a 10-in. square window during most of the time until daylight and observed in detail the destruction of the storm. Trees on the Prado were laid flat, roofs were blown off near-by buildings, and walls caved in, but the interesting thing was the pulsations of the wind. It was not a continuous blow of maximum power, but had an ebb and flow, such as possibly would be very difficult to reproduce in a wind tunnel.

[27] Cons. Engr., Chicago, Ill.

[27a] Received by the Secretary April 22, 1932.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that errors in the record be pointed out and a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from May 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

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The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

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The abbreviations in Italics represent respectively, *TT*, Total Time; *SP*, Sub-Professional Work; *P*, Professional Work; *RC*, Responsible Charge; *D*, Design. The figure for Total Time is determined by adding one-half the time spent in Sub-Professional Work to the time spent in Professional Work. The figures showing the amount of time spent in Responsible Charge and on Design are the estimate of the Applicant. The allowance of four years for graduation or of one-half of a year for each academic year successfully completed in an engineering college without graduation is included in Total Time and Professional Work.

FOR ADMISSION

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(9) **BARBEE, JOSEPH FRANKLIN,** 91 West Lakeview Ave., Columbus, Ohio. (Age 23. Born Grove City, Ohio.) 1930 B. C. E., Ohio State Univ. *TT 4: P 4.*—Aug. 1930 to date with Ohio State Highway Dept., as Inspector at gravel plant and Asst. Engr. in Testing Laboratory, testing road materials. *TT 0.9: SP 0.9.*—*TT 4.9: SP 0.9: P 4.* Refers to E. F. Coddington, L. Lee, R. R. Litehiser, C. E. Sherman.

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(13) **BECK, THOMAS GEORGE GORDON,** New Zealand. (Temporary address, Faculty Club, Univ. of California, Berkeley, Cal.) (Age 31. Born Palmerston South, Otago, New Zealand.) Aug. 1919 to Aug. 1931 with Public Works Dept., Govt. of New Zealand, until Aug. 1923 as Civ. Eng. Cadet at Auckland, Dargaville and Arthurs Pass, drafting, tracing, setting out work for construction of two railways and a railway tunnel, involving structures, embankments, cuttings, culverts, location of center line for plate laying, surveys, etc., setting out and in charge of tunnel excavation and lining, plate laying through tunnel and trackwork in station yard, setting up equipment for quarrying, crushing and screening stone ballast for 50 miles of track, setting out and constructing a railway bridge across Bealey River (seven 60-ft. steel girder spans), supervising erection of fifty cottages; Aug. 1923 to March 1924 Asst. Civ. Engr. at Otira, West Portal Otira Tunnel, directing erection of power house and machinery, transmission and pipe-lines and catenary overhead system for electric traction, conducting tests, directing operation over electrified section, etc.; March to Oct. 1924 student (selected by Dept.), Canterbury Coll. of Eng., Univ. of New Zealand; after Nov. 1924 Asst. Civ. Engr. at Dunedin, being Asst. to Engr. on design, construction and maintenance of highways (approx. 40 miles, $1 200 000), design and construction of public buildings, lighthouses, irrigation works, water-works, sewage disposal plants and drainage works; on design and construction of part of Taieri Plain flood protection works, also in executive charge of construction and maintenance of three large public institutions. *TT 10.8: SP 1: P 9.8: RC 8.3: D 6.7.*—Aug. 1931 awarded Commonwealth Fund Service Fellowship to study irrigation, flood control and land drainage in the United States; at present at Univ. of California. *TT 0.5: P 0.5: RC 0.5.*—*TT 11.3: SP 1: P 10.3: RC 8.8: D 6.7.* Refers to B. A. Etcheverry, S. T. Harding. (Applies in accordance with Sec. 1, Art. 1, of the By-Laws.)

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(15) **BERETTA, JOHN WARD,** 404 West French Pl., San Antonio, Tex. (Age 33. Born Laredo, Tex.) Licensed Prof. Engr., New York State. 1923 B. S., Mass. Inst. Tech. *TT 4: P 4.*—Nov. 1923–Nov. 1927 (except about 7 months on leave) with American Bridge Co., until Dec. 1924 at Ambridge (Pa.) Plant, inspecting railroad bridges and detailing structural steel for bridges, turntables, mill buildings, etc., and (March to June 1924)

Draftsman, Erection Dept., Pittsburgh Div., some design of erection equipment and (1 month) acted as Timekeeper and did surveying and engineering work on Kentucky Hotel, Louisville, Ky.; March to Dec. 1925 Asst. Engr., Erection Dept., Eastern Div., being Res. Engr. on suspended span of Delaware River Bridge, Philadelphia, Pa., to Camden, N. J., and in office designing and detailing erection equipment for Carquinez (Cal.) Straits Bridge, being in responsible charge of work; Feb.–May 1926 at Elmira Plant on structural steel details for New York City subway work, highway and railroad bridges, cable calculations and detail shop drawings for a suspension bridge; after June 1926 in New York office in charge of estimates and designs of steel structures. *TT 2.7: SP 0.6: P 2.1: RC 2.1: D 1.*—Dec. 1924 to March 1925 Agent, negotiating sale of properties of Texas Mfg. & Eng. Co. of San Antonio, Tex. *TT 0.2: P 0.2: RC 0.2.*—Jan. 1926 on independent steel design (2 weeks) and with U. S. Army training as Reserve Engr. Officer. *TT 0.1: P 0.1: RC 0.1: D 0.1.*—Dec. 1926 to March 1927 on design, estimates, reports and field investigations of a toll-bridge project. *TT 0.3: P 0.3: RC 0.3: D 0.1.*—Jan. 1928 to date Pres. and Active Head, J. W. Beretta Engrs., Inc., Cons. Engrs., in charge of design and supervision of Martinez St. Bridge in San Antonio, Garden St. Bridge in New Braunfels (both concrete, rigid frame), San Antonio Light Bldg., Frost Store Bldg., Groos National Bank Bldg., Roth Bros. Garage Bldg. and many others in San Antonio territory; past 3 years has acted as Res. Aeronautical Engr. for Barber & Baldwin, Inc., of New York, inspecting airplanes, airports, airways, handled adjustments, etc.; served as Cons. Engr. on appraisals and valuations, made various engineering reports, etc. *TT 4.3: P 4.3: RC 4.3: D 4.3.*—*TT 11.6: SP 0.6: P 11: RC 7: D 5.5.* Refers to E. P. Arneson, W. G. Grove, T. E. Huffman, D. E. Proper, D. B. Steinman, T. U. Taylor, J. E. Wadsworth.

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(15) BOYLE, HAROLD THOMAS, 409 Mercantile Bldg., Dallas, Tex. (Age 32. Born Osmond, Nebr.) **1923 B. S. in Agri. Eng., Iowa State Coll. *TT 4: P 4.*—June 1923 to April 1925 Rodman and Instrumentman, U. S. Reclamation Bureau, St. Ignatius, Mont., on multiple arch concrete dam and location of irrigation canals. *TT 1.5: SP 0.3: P 1.2.*—May 1925 to March 1926 Instrumentman, U. S. Bureau of Public Roads, Grangeville, Idaho, on location and construction. *TT 0.8: P 0.8.*—March 1926 to July 1928 Asst. Res. Engr., Morgan Eng. Co., Memphis, Tenn., on location and reconstruction of drainage and flood-control projects in Mississippi County, Ark., involving enlarged and new ditches (416 miles), etc. *TT 2.3: P 2.3: RC 2.3.*—July 1928 to June 1931 Asst. Res. Engr. with Myers, Noyes & Forrest, Dallas, Tex., on flood-control project, City and County of Dallas Levee Improvement Dist. ($6 500 000) involving levees, pumping plants, concrete sluiceway, drainage works, etc.; Res. Engr. on irrigation project in Starr County, Texas Water Control and Improvement Dist. No. 1, involving topographic surveys and construction of irrigation works. *TT 2.9: P 2.9: RC 2.9.*—June 1931 to March 1932 with T. A. Griffin, Gen. Contr., Dallas, as Asst. and Supt. on Maverick County Improvement Dist. No. 1, involving installation of a lateral distribution system on 18 000 acres ($65 000). *TT 0.8: P 0.8: RC 0.8.*—*TT 12.3: SP 0.3: P 12: RC 6.* Refers to Q. C. Ayres, H. W. English, T. C. Forrest, Jr., A. S. Fry, L. L. Hidinger, E. N. Noyes, N. H. Sayford.**

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(4) BRUDER, THOMAS EUGENE, 935 Morgan Ave., Drexel Hill, Pa. (Age 43. Born New York City.) **1911 B. S. in C. E., and 1925 C. E., Univ. of Pa. *TT 4: P 4.*—1911 to 1912 Surveyor, property monuments, maps, survey closing, road locations and grades on 1 000 acres to be developed into townsites, cemeteries, etc. (about 5 months), Laborer, Wm. Cramp & Co., Bldg. Contr., placing and bending reinforcing steel for 4-story reinforced concrete building, West Philadelphia High School (about 4 months), Draftsman, M. of W., Pennsylvania R. R., drawing sidings, minor highway bridges, right-of way maps, etc. (1 month), Asst. Engr., Ballinger & Perrot, Archts., on design of trusses, hotels, floor systems, etc. (under direction of Chf. Engr.) (1 month), Surveyor on 3 000-acre colonization project of Chesapeake City, Md., 1 to 4 acre plots, with appurtenant highways, maps, etc. (4 months). *TT 1: SP 0.2: P 0.8: RC 0.7: D 0.3.* 1912 to 1913 Statistician, Draftsman and Estimator for A. Merritt Taylor, Transit Commr., on preliminary report for proposed subways in Philadelphia. *TT 0.5: SP 0.5: D 0.2.*—1913 to 1922 and 1923 to 1925 with Dept. of City Transit, Philadelphia, about 6 months as Draftsman and Acting Squad Boss, in charge of men locating (from records and surveys) underground structures along proposed route of Broad St. Subway, about 10 months studying changes in surface transit system to provide feeders for subway system and to eliminate or change routes, estimates of revenue, operating expense, etc.; 1914 to 1922 Asst. Engr. and Squad Boss, on design, drawings and foundations for Frankford Elevated Ry. (4 miles), sewer designs and details**

for Central Loop Dist., Broad St. Subway sewers, pipe lines, conduits, etc.; 1923 to 1925 Asst. to Engr. of Design. *TT 10.8: P 10.8: RC 10: D 9.9.*—1922 to 1923 Asst. Chf. Engr., Ritler & Shay, Archts., on 30-story Packard Bldg., in charge (under Chf. Engr.) of structural design and drawings, checking shop drawings and field supervision of installation of foundations and bulkheads. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—1925 to date with Simon & Simon, Archts., on structural design of important buildings. *TT 7.2: P 7.2: RC 7.2: D 7.2.*—*TT 24: SP 0.7: P 23.3: RC 18.4: D 18.1.* Refers to C. Haydock, W. Linker, J. B. Myers, C. H. Stevens, S. M. Swaab.

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(15) **DALRYMPLE, TATE,** 3208 Harris Park Ave., Austin, Tex. (Age 27. Born Llano, Tex.) 1931 B. S. in C. E., Univ. of Tex. *TT 4: P 4.*—Oct. 1924 to March 1925 Rodman and Recorder, Topographic Branch, U. S. Geological Survey. *TT 0.2: SP 0.2.*—March 1925 to date (until Aug. 1931 while student) Hydrographer, Board of Water Engrs. *TT 0.7: P 0.7: RC 0.7.*—*TT 4.9: SP 0.2: P 4.7: RC 0.7.* Refers to E. C. H. Bantel, C. S. Clark, A. H. Dunlap, C. E. Ellsworth, O. A. Faris, J. A. Norris.

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(1) **DEL BOURGO, JACOB JOSEPH,** City Hall, Newark, N. J. (Age 37. Born Yokohama, Japan.) 1924 C. E., Cornell Univ. *TT 4: P 4.*—June 1924 to March 1926 and Oct. 1928 to April 1929 Structural Draftsman and Checker, Board of Transportation, New York City, drafting, designing and checking structural drawings for subways and appurtenant structures. *TT 2.2: P 2.2: RC 0.7.*—March to Dec. 1926 Structural Draftsman, Dwight P. Robinson & Co., Inc., New York City, drafting and designing reinforced concrete structures. *TT 0.7: P 0.7.*—Dec. 1926 to Nov. 1927 Structural Draftsman and Acting Chf. Draftsman, Knickerbocker Ice Co., New York City, designing and checking buildings for ice-manufacturing plants. *TT 0.9: P 0.9: RC 0.6.*—Nov. 1927 to Oct. 1928 Structural Engr., New York City, designing steel for architects. *TT 0.9: P 0.9: RC 0.9.*—April 1929 to April 1930 Asst. Engr., Culver Contr. Corporation, Brooklyn, N. Y., supervising subway construction, keeping records, preparing bids, assisting Chf. Engr. *TT 1: P 1: RC 1.*—May 1930 to date Asst. Engr., Transit Bureau, Newark, N. J., designing, checking, estimating, supervising draftsmen on city railway, preparing structural drawings for railway and other structures. *TT 2: P2: RC 2.*—*TT 11.8: P 11.8: RC 5.2.* Refers to H. G. Babcock, J. Feld, R. Smillie, E. Welle, L. S. Whippen.

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(9) **ECKERLE, WILLIAM PETER,** 326 West Main St., Frankfort, Ky. (Age 31. Born Louisville, Ky.) 1923 C. E., Univ. of Notre Dame. *TT 4: P 4.*—Feb. 1923–Oct. 1930 with Illinois Central R. R., until Nov. 1925 successively as Chainman, Rodman and Instrumentman, then Masonry Inspector, Bridge Dept., in responsible charge of construction of reinforced concrete, steel and timber bridges and of lining a 6 985-foot tunnel with concrete, on construction of heavy piers, abutments, steel viaducts, encasement of steel, bridge investigation, design of forms, and of concrete and timber bridges. *TT 6.7: SP 1.2: P 5.5: RC 5.5: D 2.*—Feb. 1931 to date with Kentucky State Highway Dept. Bridge Div., 1 year as Designer, 2 months writing specifications for road and bridge work, and since April 1932 Asst. Bridge Engr. *TT 1.3: P 1.3: RC 1.2: D 1.*—*TT 12: SP 1.2: P 10.8: RC 6.7: D 3.* Refers to H. R. Creal, P. D. Gillham, J. T. Madison, E. D. Smith, W. S. Todd, F. G. Vent, C. C. Westfall.

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(13) **EDGECOMBE, ARTHUR RALPH BROOKS,** Faculty Club, Univ. of California, Berkeley, Cal. (Age 29. Born Bewdley, Worcester, England.) 1923 B. Sc. (Eng.), Univ. of London. *TT 4: P 4.*—Aug. 1923 to Aug. 1924 Asst. to D. Edwards, City Engr., Bath, England, on surveys and estimates for New Southdown Road ($70 000), replanning city areas at Dolemeads. *TT 1: P 1: D 0.5.*—Aug. 1924 to Dec. 1929 Asst. Executive Engr., and Dec. 1929 to July 1931 Acting Executive Engr., with Indian Service of Engrs., Govt. of India, engaged as follows: Aug. 1924 to March 1925 at Shillong, Assam, on surveys and estimates for water supply to cantonments; March to Oct. 1925 Officer, Headworks Constr. Div., Sarda Canal, United Provinces, on details of design and supervision of labor; Oct. 1925 to Nov. 1926 in charge of Bahadurabad Temporary Subdivision, Upper Ganges Canal, reconditioning and enlarging hydro-electric station, installing Diesel engine stand-by plant, erecting and modernizing 20 miles high-tension line, design and erection of substations ($150 000), Nov. 1926 to Aug. 1927 in charge of second subdivision, Northern Div., Ganges Canal, control of irrigation district (approx. 80 000 acres), approx. 230 miles of canal and appurtenant works, Aug. 1927 to Nov. 1928 on preliminary surveys and designs for Ramganga Canal headworks and two electrically operated pumping stations

(approx. $332 000) ; Nov. 1928 to Dec. 1929 Subdivisional Officer, Ramganga, hydroelectric scheme, on design, estimate and supervision of construction of Mahmudpur lateral canal ($100 000), supervising erection of 200 miles of high-tension line ($350 000) and reserve oil operated power house ($120 000) ; Dec. 1929 to Oct. 1930 in charge of Ken Canal Div. (850 000 acres), Banda, comprising storage works (approx. 50 000 acre-ft. and approx. 400 miles of canal, after Oct. 1930 in charge of Jhansi Div., Betwa Canal, storage works (approx. 200 000 acre-ft.), supervising preliminary surveys and designs for Jamni River storage scheme to compound 160 000 acre-ft. of water. *TT 7.1: P 7.1: RC 5.3: D 2.6.*—Aug. 1931 awarded Harkness Commonwealth Fellowship to study (2 years) irrigation and allied works in United States. *TT 12.1: P 12.1: RC 5.3: D 3.1.* Refers to B. A. Etcheverry, S. T. Harding, F. C. Herrmann, C. G. Hyde, F. C. Scobey.

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(1) **FITZPATRICK, EDWARD BERNARD,** 1061 Grant Ave., Bronx, N. Y. (Age 36. Born New York City.) 1924 B. S. in C. E., Cooper Union Inst. Tech. *TT 4: P 4.*—June 1924 to June 1926 with Erie R. R., successively as Rodman, Levelman and Transitman on surveys, preliminary studies, designs, investigations, plans, estimates and staking out yards, track and right-of-way, part of time in charge of work under Chf. of Engr. Corps. *TT 1.7: SP 0.2: P 1.5: RC 1.5: D 0.3.*—June 1926 to Sept. 1927 Chf. Designer and Res. Engr., Elec. Ferries, Inc., on four ferry terminals, in charge of design and construction of roads, bridges, viaducts, ferry bridges and racks, docks, bulkheads, sewers, walls and small buildings. *TT 1.3: P 1.3: RC 1.3: D 0.5.*—Sept. 1927 to Nov. 1928 Eng. Draftsman, New York Central R. R., New York City, on New York Central Bldg., being Asst. on structural designs and details, checking architects' plans and contractors' proposals. *TT 1.2: P 1.2: D 0.5.*—Nov. 1928 to date Asst. Engr. (Structural Design), Westchester County Park Comm., in charge (under Designing Engr.) of making and checking preliminary studies, designs, investigations, plans, specifications, estimates and shop details on concrete and steel highway bridges, including square and skew rigid frames, arches, cantilevers, etc. *TT 3.4: P 3.4: RC 3.4: D 2.*—*TT 11.6: SP 0.2: P 11.4: RC 6.2: D 3.3.* Refers to J. Barnett, T. Fenner, H. F. Finck, A. G. Hayden, L. G. Holleran, R. M. Hodges, W. F. S. Root, B. L. Weiner.

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(1) **FONT, GILBERTO MELQUIADES,** 24 Miramar Ave., Santurce, Porto Rico. (Age 29. Born Aguadilla, Porto Rico.) 1926 B. S. in C. E., Univ. of Mich. *TT 4: P 4.*—June 1926–Nov. 1928 with Isabela Irrigation Service, until March 1927 as Instrumentman on Guajataca Dam, in charge of grades, lines, estimates, cost analysis and inspection of concrete work, then Asst. Engr., Distribution System, in charge of camp construction, transportation of materials by government trucks and building of concrete structures.—*TT 2.3: P 2.3.*—Dec. 1928–Oct. 1929 Asst. Engr., Rio Blanco Power Development, Eng. Dept., Porto Rico Ry. Light & Power Co., in charge of design of structurals, surveying and inspection work. *TT 0.8: P 0.8: RC 0.8: D 0.4.*—Dec. 1929 to date Supt. of Constr., with U. S. Army, 1st C. Smith, Constr. Q. M. C., in charge of construction of Co. Officers' and Field Officers' Quarters, General Headquarters Bldg. and on design of sewers, roads and water lines, also on design and construction of $10,000 private residence. *TT 2.3: P 2.3: RC 2.3: D 1.*—*TT 9.5: P 9.5: RC 3.1: D 1.4.* Refers to E. Baez Rodriguez, C. A. Garcia, R. A. Gonzalez, R. Ramirez, R. M. Snell, E. Totti y Torres.

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(9) **FRASCH, BURROWS HOLCOMB,** 129 West Main St., Newark, Ohio. (Age 37. Born Corning, Ohio.) 1917–1924 (except part of 1918 with Air Service, U. S. Army) Petroleum Engr., about 4 months with Empire Gas & Fuel Co., in Montana and Wyoming, about 4 months with National Refining Co. in Kansas, 5½ years with Standard Oil Co. of New Jersey, in Venezuela, Colombia, Bolivia, Argentina, Paraguay and Brazil, and 6 months with Carter Oil Co., in Colorado and Utah, on preliminary, detail and valuation work, running planetable, mapping drainage and topography, preparing plans and reports, locations in unsurveyed and little-known parts of South America, determining road and pipe line locations, etc. *TT 6.5: SP 0.1: P 6.4: RC 5: D 1.2.*—1925 Vice-Pres. and Part Owner, Newkirk Eng. Co., Winterhaven, Fla., and 1926 to 1929 Pres. and Owner, Birmingham (Ala.) Engrs., Inc., private and consulting engineering, preparing plans and specifications, and designing and supervising construction projects, subdivision designing, city planning, road and street paving, designing and constructing golf-course, small dam, sanitary and storm sewers, reclamation, drainage, etc. *TT 5: P 5: RC 3.8: D 1.2.*—1929 City Engr., Hollywood, Ala., in charge of street paving, sanitary and storm sewers. *TT 1: P 1: RC 1.*—1930 to date with Ohio State Highway Dept., 9 months as Engr.-Inspector, Div. No. 12, in

charge of construction projects, and since 1931 Res. Div. Deputy Director at Newark, Ohio, in charge of maintenance, plans, designs on roads and bridge and of construction work in Div. No. 5. *TT 2.8: P 2.8: RC 2.8.*—*TT 15.3: SP 0.1: P 15.2: RC 12.6: D 2.4.* Refers to C. Ash, B. D. Greenshields, R. E. Hamilton, T. S. Johnson, R. H. Smith.

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(7) **GAUCKLER, ANTHONY JOSEPH,** 2232 North Twenty-ninth St., Milwaukee, Wis. (Age 34. Born Milwaukee, Wis.) 1919 A. B., 1920 B. S. in C. E., and 1923 M. A., and C. E., Marquette Univ. *TT 4: P 4.*—Sept. 1920 to Sept. 1922 Instructor in Civ. Eng., Marquette Univ. *TT 2: P 2.*—May to Oct. 1923 Structural Engr. with Hool, Johnson & Whitney, structural design of buildings. *TT 0.4: P 0.4: D 0.4.*—Oct. 1923 to March 1925 Archt.'s Supt., with Eschweiler & Eschweiler, superintending construction of two large school buildings. *TT 1.4: P 1.4: RC 1.4.*—Jan. 1927 to Sept. 1930 Structural Engr., until April 1929 with C. Hennecke Co., then with Osthoff-Peterson Co., Inc., in charge of structural steel and general structural design. *TT 3.6: P 3.6: RC 3.6: D 3.6.*—March 1925 to Jan. 1927 and Sept. 1930 to date Structural Engr., Milwaukee (Wis.) School Board, in charge of structural design of school buildings. *TT 3.5: P 3.5: RC 3.5: D 3.5.*—*TT 14.9: P 14.9: RC 8.5: D 7.5.* Refers to J. D. Bonness, B. E. Brevik, J. P. Gebhard, O. P. Osthoff, L. E. Peterson, E. D. Roberts, C. S. Whitney.

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(1) **GOLDBERGER, HAROLD WILLIAM,** 43-47 One Hundred Fifty-ninth St., Flushing, N. Y. (Age 26. Born New York City.) 1931 B. S. in C. E., N. Y. Univ. *TT 4: P 4.*—July 1927 to date on subway construction in New York City, until April 1930 as Jun. Engr. for Oakdale Contr. Co., Inc., on Route 101, Sec. 5, transit and level work, etc., then (1 year) Chf. of Field Party for Rosoff Subway Constr. Co., Inc., on Route 107, Sec. 10, in responsible charge of lines, grades, computations, etc., and since April 1931 Constr. Engr. for Cornell Contr. Corporation on Route 107, Sec. 8, designing and expediting work. *TT 0.6: P 0.6: RC 0.6: D 0.5.*—*TT 4.6: P 4.6: RC 0.6: D 0.5.* Refers to A. H. Diamant, J. H. C. Gregg, B. A. Hodgdon, C. M. Madden, L. C. Pope, M. T. Staples, F. W. Stiefel.

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(14) **GUERNSEY, CURTIS HAROLD,** 1216 Grand, Cherokee, Okla. (Age 39. Born McCraken, Kans.) 1912 to 1917 Draftsman, successively with W. F. Corbin, and Boller Bros., Archts., Kansas City, Mo. *TT 2.5: SP 2.5.*—1917 to 1920 in military service.—1920 to 1925 County Agt., Extension Div., Oklahoma Agricultural and Mechanical Coll. on reclamation of farm lands, stream and erosion control, etc., in Custer and Payne Counties, Okla.—*TT 4: SP 1: P 3: RC 3: D 3.*—1925 to 1927 Levelman, Instrumentman and Draftsman with locating party, Oklahoma State Highway Dept. *TT 1: SP 1.*—1927 to date County Engr., Alfalfa County, Okla., and City Engr., Cherokee, Okla.; work has included responsibility for design and supervision of construction of Cherokee City Paving Dists. Nos. 4 and 5 ($140 000) and storm sewers and water-works extensions ($50 000), Medicine River Levee Dist. No. 1, Alfalfa County ($40 000), Pond Creek Drainage Dist. No. 9, Grant County, Okla., for State Highway Comm. and Grant County ($53 000), also had charge of highways, bridges, street grade, drainage, etc. *TT 5: P 5: RC 5: D 5.*—*TT 12.5: SP 4.5: P 8: RC 8: D 8.* Refers to E. S. Alderman, S. A. Hott, R. V. Lindsey, J. B. Marcellus, C. H. Rightmire.

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(9) **HALLOCK, HARRY EARNEST,** 40 Parkwood Ave., Columbus, Ohio. (Age 23. Born Columbus, Ohio.) 1930 B. C. E., Ohio State Univ. *TT 4: P 4.*—June 1931 to date Inspector, Ohio State Highway Dept., Bureau of Tests, on asphalt inspection and testing materials in laboratory. *TT 0.4: SP 0.4.*—*TT 4.4: SP 0.4: P 4.* Refers to E. F. Coddington, L. Lee, R. R. Litehiser, C. E. Sherman.

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(13) **HERSEY, EDWIN SPAULDING,** 302 North Division St., Carson City, Nev. (Age 27. Born Randlett, Utah.) Student, Univ. of Cal. (Oct. 1924-May 1925) and Riverside Jun. Coll. (Oct. 1928-May 1930). *TT 0.5: P 0.5.*—May 1922 to Sept. 1924 Chainman and Inspector, U. S. Indian Service, on construction of roads and timber bridges, Ute, Indian Reservation. *TT 1.2: SP 1.2.*—May 1925 to Sept. 1928 Chainman, Levelman and Instrumentman, Southern California Edison Co., on construction of multiple arch dam, tunnels, siphon and penstock lines, in charge of parties. *TT 3: SP 0.5: P 2.5: RC 0.5.*—Summer 1929 with Southern Sierras Power Co., estimating power-line and sub-station construction. —June 1930 to March 1931 Asst. Res. Engr., California State Highway Com., on con-

struction of asphaltic concrete and pre-mix oil-surface roads. *TT 0.8: P 0.8: RC 0.8.*—April 1931 to date Engr., Nevada State Highway Dept., in charge of construction of county road-mix oil roads and Chf. Inspector on asphaltic-concrete highways, and (winter months) draftsman on design. *TT 1: P 1: RC 0.7.*—*TT 6.5: SP 1.7: P 4.8: RC 2.*—Refers to R.. C. Booth, W. L. Chadwick, G. R. Egan, C. L. Hill, F. B. Laverty, R. W. Spencer.

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(7) **HICKMAN, HAROLD CLARENCE,** 12026 Manor Ave., Detroit, Mich. (Age. 24. Born Ft. Wayne, Ind.) 1931 B. S. in Civ. Eng., Univ. of Mich. *TT 4: P 4.*—July 1924 to Sept. 1929 (while student) Office Man, Detroit (Mich.) Edison Co., office routine and construction work, concrete research, surveys and reports.—Summer 1930 Jun. Engr. (Inspector rating), U. S. Dist. Office, Vicksburg, Miss., reporting hydrological features of Red River, determining runoff from drainage basins and general hydraulic engineering problems.—Feb. 1932 to date Federal Highway Engr., Bureau of Public Roads, on cost and production studies. *TT 0.1: SP 0.1.*—*TT 4.1: SP 0.1: P 4.* Refers to T. W. Allen, G. R. Clemens, H. W. King, C. O. Wisler, J. S. Worley.

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(1) **HOLT, ARTHUR WINSTON,** 86–09 Elmhurst Ave., Elmhurst, N. Y. (Age 28. Born Charlottesville, Va.) 1926 C. E., Univ. of Va. *TT 4: P 4.*—Summer 1922 with Bell Telephone Co., Trenton, N. J., and summer 1923 with Appalachian Power Co., Bluefield, W. Va., on service work.—June 1926 to Jan. 1932 with Bartlett Hayward Co., Baltimore, Md., as Jun. Engr. and Field Supt. in charge, on construction and operation of gas plants and gas holders in Toronto, Detroit, Los Angeles, San Francisco, etc. *TT 5.5: P 5.5: RC 3.*—*TT 9.5: P 9.5: RC 3.* Refers to T. B. Kiener, H. M. Lloyd, R. C. Marshall, Jr., J. L. Newcomb, J. M. Webster.

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(16) **KEPLEY LE ROY FRANCIS,** Chanute, Kans. (Age 22. Born Chanute, Kans.) 1931 B.S. in C.E., Kans. State Coll. *TT 4: P 4.*—May to Sept. 1931 Slab Inspector, and Sept. 1931 to date Plan Draftsman, Kansas State Highway Comm. *TT 0.9: P 0.9: RC 0.6.*—*TT 4.9: P 4.9: RC 0.3: D 0.6.* Refers to H. D. Barnes, L. E. Conrad, O. J. Eidmann, F. F. Frazier, M. W. Furr, C. H. Scholer.

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(12) **KRABBE, JOHAN ADOLPH,** 519 Thirteenth St., Bellingham, Wash. (Age 25. Born Nazareth, Pa.) 1931 B. S. in Civ. Eng., State Coll. of Wash. *TT 4: P 4.*—Sept. 1929 to Sept. 1930 with Olympic Portland Cement Co., Ltd., Bellingham, Wash., 3 months as Asst. Constr. Engr. surveying, building forms for concrete, pouring concrete and placing reinforcing steel and machinery, and 8 months Timekeeper, ordering materials, acting as Draftsman, etc., on a limestone-crushing plant, and after Aug. 1930 Inspector on construction of concrete silos, inspecting placing of reinforced steel, pouring concrete and form work. *TT 0.6: SP 0.3: P 0.3: RC 0.3: D 0.2.*—June 1931 to date Chainman, Washington State Highway Dept., drafting, earthwork calculations, inspection, etc. *TT 0.4: SP 0.4.*—*TT 5: SP 0.7: P 4.3: RC 0.3: D 0.2.* Refers to H. E. Phelps, M. K. Snyder, J. G. Woodburn.

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(1) **LANE, JOHN STERLING,** Casilla 15, Tocopilla, Chile. (Age 29. Born San Francisco, Cal.) 1926 B. S. in C. E., Mich. State Coll. *TT 4:P 4.*—March to Oct. 1925 Asst. Field Engr., H. G. Christman Co., Detroit, Mich,, acting as Chainman and Rodman and (4 months) in charge of field party on industrial building layout for excavation, foundations, form and brick setting, lines and levels.—June 1926 to Dec. 1927 and July 1928 to May 1929 with Everett Winters Co., Detroit, until Dec. 1927 as Field Engr. and Asst. Supt. on layout of commercial buildings, lines and levels, excavations, foundations, forms, brick and steel, form design, reports, sub-contracts, and after July 1928 Chf. Field Engr. and Asst. Supt., at Saginaw, Mich., in charge of field parties and subcontractors on dock and industrial building construction, property checks, excavations, pile and spread foundations, forms, etc. *TT 2.3: SP 0.1: P 2.2: RC 2.1: D 0.6.*—Dec. 1927 to July 1928 Field Engr. and Asst. Supt., Christman-Burke Co., Detroit, plan checking and correcting, form design, property-line check, building layout for excavation, foundations, forms, iron and ornamental work, reports. *TT 0.5: P 0.5: RC 0.5: D 0.2.*—Aug. 1929 to Jan. 1930 with Wurster Constr. Co., Los Angeles, Cal., as Field Engr. and Asst. Supt. at Agua Caliente, Mexico, race-track building layouts, stables, grandstand, club, paddock, etc., lines, levels, plan checks, forms, brick and ornamental work. *TT 0.5: P 0.5: RC 0.4.*—Feb. to April 1930 Field Engr., Healy-Tibbitts Constr. Co., San Francisco, on pile and concrete layout for Carquinez Bridge pier buffer, material accounts, time, etc.—*TT 0.2: SP 0.1:*

P 0.1: RC 0.1.—April 1930 to date Field Engr., Lautaro Nitrate Co., Ltd., New York City, on nitrate oficina construction in Chile, topography, triangulation, monuments, etc., layouts for rock excavations, foundations, bolts, etc., concrete anad structural steel buildings, retaining walls, fills, roads, sewers, railroads, concrete trenches, quantity surveys, reports, inspection. *TT 1.8: SP 0.1: P 1.7: RC 1.4: D 0.2.*—*TT 9.3: SP 0.3: P 9: RC 4.5: D 1.* Refers to C. L. Allen, C. M. Cade, J. D. Galloway, J. P. Marshall, R. A. Murdoch, H. C. Rapp.

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(14) **LIVINGSTON, JOHN JOSEPH,** 404 East Fourth St., Rolla, Mo. (Age 26. Born in Phelps County, Mo.) 1932 B. S. in Min. Eng., Mo. School of Mines. *TT 4: P 4.*—June to Dec. 1928 and Feb. to Oct. 1929 Asst. Civ. Engr., with United Fruit Co., Colombia, designing, drafting, surveying, drainage and irrigation (1928) and with Edward Flad & Co., Cons. Engrs., St. Louis, Mo., designing, inspecting and supervising construction (1929). *TT 1.1: SP 0.1: P 1: RC 0.7: D 0.5.*—Oct. 1929 to June 1930 Designing Draftsman, Wabash Ry. Co., St. Louis, on bridge design. *TT 0.4: SP 0.4.*—June 1930 to Aug. 1931 Chf. of Party, Stone & Webster Eng. Corporation, St. Louis, on location, level, land line and construction parties on Bagnell Dam transmission lines. *TT 0.9: SP 0.3: P 0.6.*—At present graduate student, Missouri School of Mines.—*TT 6.4: SP 0.8: P 5.6: RC 0.7: D 0.5.* Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris, C. V. Mann.

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(1) **MACKENZIE, LEONARD ALEXANDER,** P. O. Hartebeestpoort, via Brits, Transvaal, South Africa. (Age 39. Born Ladysmith, Natal, South Africa.) 1917 diploma in Civ. Eng., South African Coll. 1918 B. S. in Civ. Eng., Univ. of Cape Town.—1914 to 1919 War service.—June 1919 to date with Dept. of Irrigation, Union of South Africa, until Sept. 1924 as Asst. Engr., about 10 months on preliminary survey of Orange River, location, levelling and topography of main tacheometric traverses, chiefly on triangulation of 50-mile reconnaissance, about 9 months in Head Office of Dept., on design for various projected schemes, including alignment and dimensions of two main canals (each about 50 miles) of Hartebeestpoort project, on design and specifications of steel and reinforced concrete siphon, design of 200-ft. spillway for Lake Mentz project, checking calculations of variable radius arch dam across the Crocodile River, drawing up tenders, specifications, estimates, contracts, alterations in and design of minor irrigation works, 1 2/5 years with Southern Transvaal Circle, on advisory cases, consisting of survey, design and estimates of farm irrigation projects, some supervision, also on survey of basin, damsite, and location of 15 miles of main canal, design of canal and works of Commissie Drift project, survey of 16 miles of main canals, design of earth embankment and complete design of Schweizer Reneke Project, 4 months measuring, setting out and supervising work on Lake Mentz project, and 2 years on Van Rynevelds Pass project, responsible for construction of mass concrete wall (140 000 cubic yards), design and construction of filtration plant and irrigation system for Town of Graaff Reinet and various structures of main project; Oct. 1924 to Aug. 1925 Res. Engr. on completion of this project (cost £400 000); Aug. 1925 to April 1926 in Head Office of Dept. on design work, and (3 months) on reconnaissance survey of Pongolo River, locating 40 miles of canal; May 1926 to May 1929 Senior Asst. Engr., Southern Transvaal Circle, on farm advisory cases, collecting hydrographic data, being Engr. Assessor to Water Court and giving professional advice to Depts. of Mines, Agriculture and Native Affairs, also on design and supervision of minor irrigation works; since May 1929 Administration Engr., Hartebeestpoort Irrigation Project (40 000 acres). *TT 12.6: P 12.6: RC 5.5: D 11.4.* Refers to F. W. Scott, W. G. Sutton, R. J. van Reenen. (Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

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(1) **McINTOSH, WALTER TOWNSEND,** 15 Park St., Tenafly, N. J. (Age 46. Born Buffalo, N. Y.) Licensed Prof. Engr. and Land Surveyor, New York State. 1907 B. E., Union Coll. *TT 4: P 4.*—Aug. 1907 to Oct. 1919 with Board of Water Supply, New York City, on Catskill Aqueduct, until April 1909 as Rodman, Instrumentman, Computer and Draftsman on surveys, plans, etc.; April 1909 to Dec. 1911 Asst. Engr., and Dec. 1911 to Dec. 1913 Asst. Sec. Engr., in responsible charge of line and surveys, measurements and computation of payment estimates for contract work, drafting, etc., on location and construction of part of aqueduct; Dec. 1913 to Sept. 1916 Office Engr., and after Sept. 1916 Sec. Engr., New York City, checking designs for parts of aqueduct, preparing and in charge of payment estimates, reports, etc., also in responsible charge of supervision of construction and of installation and operation of mechanical equipment of aqueduct; 1911 to 1916 also made several property surveys, for Orange County, Storm King Realty Corporation, etc. *TT 11.3: SP 0.8: P 10.5: RC 10.5: D 1.4.*—Oct. to Dec. 1919 Sales Engr.

and Designer, Storck Contrs. Export Corporation, New York City, designing and selling industrial housing for export, to Cuban sugar plantations, etc. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—Dec. 1919 to Feb. 1927 Sales Engr., Frazar & Co., New York City, first as Asst., then in charge of department selling steel products, construction machinery and equipment for export and later in charge of all exports to Europe, South Africa, etc.; made preliminary designs for industrial plants, etc. *TT 7.2: P 7.2: RC 7.2: D 1.*—Feb. 1927 to date Engr., Spencer, White & Prentis, Inc., New York City, in responsible charge of line and grade surveys for construction operations, principally building foundations, estimating costs, bidding and securing contracts for foundation construction work, occasionally acting as Supt. in charge of foundation construction jobs; later in charge of design of foundations, needling and underpinning for numerous buildings; designed needles and underpinning for construction of Abraham & Straus Dept. Store, Brooklyn, N. Y., Bank of Manhattan Bldg., New York City, etc.; Temporary Gen. Supt. in charge of all construction; Consultant for firm on foundation and underpinning work. *TT 5.1: P 5.1: RC 5.1: D 3.1.*—*TT 27.8: SP 0.8: P 27: RC 23: D 5.7.* Refers to W. B. Hunter, H. T. Immerman, J. Meltzer, T. Merriman, E. A. Prentis, J. F. Sanborn, L. White.

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(2) **MINAMI, JOHN KAZUO,** Mass. Inst. Tech. Dormitory, Cambridge, Mass. (Age 24. Born Seattle, Wash.) 1931 S. B., Mass. Inst. Tech. *TT 4: P 4.*—At present graduate student, in Civ. and San. Eng., Massachusetts Inst. of Technology.—*TT 4: P 4.* Refers to W. M. Fife, W. C. Voss.

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(10) **MOORE, PHILLIPS,** 25 West Washing St., Newnan, Ga. (Age 28. Born Texas, Ga.) 1924 B. S. in C. E., Ala. Pol. Inst. *TT 4: P 4.*—June 1924 to March 1925 Transitman, Watson & Garris Eng. Co., Miami, Fla., surveying and street improvement work. *TT 0.4: SP 0.4.*—March to Nov. 1925 Chf. of Party, Biscayne Eng. Co., Miami, surveying, river and harbor work, street improvement and canals. *TT 0.7: P 0.7.*—Dec. 1925 to March 1927 member of firm, Ball & Moore Eng. Co., Miami, in charge of field work, surveys and sub-division work. *TT 1.3: P 1.3.*—April 1927 to Feb. 1928 Asst. Res. Engr., Georgia State Highway Dept., Cordele, Ga., on concrete and limestone road construction. *TT 0.7: SP 0.1: P 0.6.*—March to July 1928 Chf. of Party, Atmospheric Nitrogen Corporation, Hopewell, Va., building and pier construction and railroad surveys. *TT 0.4: P 0.4.*—July 1928 to April 1931 Field Engr. and Chf. Engr., American Tel. & Tel. Co., Philadelphia, Pa., on location, in complete charge of underground conduit construction. *TT 2.8: P 2.8: RC 2.*—May 1931 to date Transitman and Asst. Res. Engr., State Highway Dept. of Georgia, Atlanta, Ga., grading and concrete road construction, also brick paving. *TT 0.8: SP 0.2: P 0.6.*—*TT 11.2: SP 0.8: P 10.4: RC 2.*—Refers to J. W. Barnett, J. D. Brown, B. P. McWhorter, E. N. Seymour, S. B. Slack, C. W. Wright.

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(7) **PATITZ, WALTER ERNEST,** 732 North Seventeenth St., Milwaukee, Wis. (Age 31. Born Park Falls, Wis.) 1925 C. E., Marquette Univ. *TT 4: P 4.*—June to Dec. 1925 Engr., Nickell-Grahl Constr. Co., on highway relocation (8½ miles) near Stroudsburg, Pa., in charge of setting grades, bridge layout, including 75-ft. span plate girder, 65-ft. span concrete arch, etc. *TT 0.5: P 0.5.*—Dec. 1925 to date with Wisconsin Highway Comm., Milwaukee, Wis., until April 1926 on computation and general plans, then Res. Engr. on construction, setting grade and line, general bridge layout, supervising inspectors, estimates and progress reports, and since Dec. 1927 Asst. Engr. in charge of materials and some construction, investigating local pit material, in charge of location maps and plans, concrete material, metal and concrete pipe, design and control of concrete mixes, supervising bridge and paving projects, soils investigation, marsh soundings and treatment. *TT 6.3: P 6.3: RC 4.4.*—*TT 10.8: P 10.8: RC 4.4.* Refers to J. D. Bonness, H. J. Kuelling, G. W. Langley, Jr., E. D. Roberts, C. R. Weymouth.

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(9) **PAULUS, RAYMOND LAWRENCE,** Rensselaer, Ind. (Age 31. Born Milwaukee, Wis.) 1922 B. S. in Elec. Eng., Univ. of Wis. *TT 4: P 4.*—June 1922-Oct. 1930 with Allis Chalmers Mfg. Co., until June 1924 as Student Apprentice in Hydraulic Research Laboratory, then on hydraulic engineering, designed hydraulic runners, draft tubes, guide vanes, etc., made field tests of hydraulic units, pumps, etc., having charge of tests on pump at River Falls Unit (15 m. g. d.) of Indianapolis Water Power Co.; after 1928 in charge of Canadian work for Hydr. Dept. and had charge of design estimating, selling and supervising manufacture of all turbines sold in Canada and Newfoundland; also some consulting work, etc. *TT 7.3: SP 1: P 6.3: RC 1: D 1.*—Nov. 1930 to date Engr., W. C. Babcock

Grain Co., designing, estimating and supervising field operations in connection with grain elevators and contracting on open-drainage ditches, roads, bridges and sewage-disposal plants, etc. *TT 1.5: P 1.5: RC 1.5.*—*TT 12.8: SP 1: P 11.8: RC 2.5: D 1.* Refers to L. E. Bogen, F. Nagler, E. D. Roberts, C. M. Scudder, C. N. Ward, W. M. White.

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(14) **PHILLIPS, CHARLES CECIL,** Cairo, Ill. (Age 29, Born Rockdale, Tex.) 1924 B. S. in C. E., Tex. Agri. & Mech. Coll. *TT 4: P 4.*—July 1924 to date with U. S. Engr. Office, until Oct. 1924, Jan. to June and Sept. to Dec. 1925 as Inspector and Asst. Chf. of Party, on Insley Tower paving bank, Vicksburg, Miss., and Willow Mat construction; Nov. to Dec. 1924 in charge of revetment party placing rip-rap on upper bank; July to Aug. 1925 Chf. of Party, making borings and taking soil samples on Mississippi River; Jan. 1926 to March 1927 on design and improvement work in connection with dredge and concrete mixing plant, Greenville, Miss.; April to Sept. 1927 Chf. Asst. to Area Engr., during flood highwater fight and after June in charge of steamboat operation; Oct. to Nov. 1927 in charge of two hydraulic graders; Dec. 1927 to July 1928 Asst. to Area Engr. in responsible charge of four survey parties and inspection and design of concrete mixtures at concrete casting plant; Aug. 1928 attended special school of Portland Cement Association relative to design and control of concrete mixtures; Sept. to Dec. 1928 designing, supervising and inspecting construction of concrete casting plant and steel forms; Jan. 1929 to Feb. 1930 Asst., in charge of Revetment Sec. Dist. Office, Vicksburg, revetment plans, sub-projects and finance; March 1930 to June 1931 Chf. Asst. to Area Engr., Central Area, Greenville, and in responsible charge of revetment construction in Vicksburg Dist.; July 1931 to date Chf. of Operations, First Field Area, Memphis Dist., Cairo, Ill., in responsible charge of revetment parties, dredging and levee construction. *TT 7.7: P 7.7: RC 4.8: D 0.5.*—*TT 11.7: P 11.7: RC 4.8: D 0.5.*—Refers to J. S. Allen, W. E. Elam, T. T. Knappen, J. C. H. Lee, T. A. Munson, J. J. Richey, B. B. Somervell.

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(9) **PRITCHARD, SAMUEL COPELAND,** 200 East Oakland Ave., Columbus, Ohio. (Age 23. Born Springfield, Ky.) 1932 B. C. E., Ohio State Univ. *TT 4: P 4.*—Summer 1928 Recorder, and Jan. to April 1929 (while student) Computer, Water Resources Branch, U. S. Geological Survey.—April 1929 to Jan. 1930 Levelman, Engr. Corps, Erie R. R., Marion, Ohio, on surveys, drafting, track layouts, plans and estimates of general maintenance work. —Jan. to June 1930 Draftsman, Office of City Engr., Columbus, Ohio, detail drawings and tracings for reinforced concrete sewage tanks.—*TT 4: P 4.* Refers to R. T. Regester, C. E. Sherman, R. C. Sloane.

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(14) **RAYMOND, ALLAN TEDFORD,** 304 Belvedere Apartments, Columbia, Mo. (Age 28. Born Boston, Mass.) 1926 B.S. in C.E., Okla. Agri. and Mech. Coll. 1931 C.E., Iowa State Coll. *TT 4: P 4.*—June 1926 to June 1927 Senior Eng. Aid, Massachusetts Highway Dept., drafting, etc. *TT 0.5: SP 0.5.*—Summer 1928 Senior San. Aid, Massachusetts Dept. of Public Health, on engineering work.—Sept. 1927 to date Instructor, Univ. of Missouri, in charge of instruction in engineering drawing and descriptive geometry. *TT 4.3: P 4.3: RC 4.3.*—*TT 8.8: SP 0.5: P 8.3: RC 4.3.*—Refers to F. M. Dawson, A. H. Fuller, W. E. Galligan, A. L. Hyde, E. J. McCaustland, H. K. Rubey, R. G. Tyler.

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(13) **ROBBINS, HARLAN ROWE,** State Bldg., San Francisco, Cal. (Age 38. Born Owosso, Mich.) 1916 B. S., Univ. of Utah. *TT 4: P 4.*—July 1916 to July 1918 Laboratoryman and (6 months) Laboratory Foreman, Gen. Eng. Co., Salt Lake City, Utah. on research, tests, etc. *TT 2: P 2: RC 0.5.*—Aug. to Dec. 1918 Private, Signal Corps, U. S. Army, at Officers' Training Camp, Yale Univ., taking Elec. Eng. course.—Jan.-June 1919 Shift Boss, Western Utah Copper Co., Salt Springs, Utah, revamped mill, installed several concentrators, etc. *TT 0.5: P 0.5: RC 0.5.*—July-Dec. 1919 Utility Foreman, C. F. Dinsmore & Son, Contrs., Ogden, Utah, on small steel construction, installing irrigation units, surveying, etc. *TT 0.5: P 0.5: RC 0.5.*—Jan. to June 1920 Shift Boss, Vipont Silver Min. Co., Oakley, Idaho, on 250-ton concentrating (flotation and gravity) mill, started mill, ran 12-hour shift, etc. *TT 0.5: P 0.5: RC 0.5.*—July 1920 to date with California R. R. Comm., 9 months as Clerk, 2 years Jun. Asst. Engr., and remainder of time Asst. Engr. on valuations, last 6 years (more than one-half of time) in responsible charge; made investigations, wrote and signed reports, was witness in court cases, etc., for water and gas works, land, water, transportation, navigation, telephone, telegraph, electric railroad, light, power, refrigeration, canal, irrigation and other companies (named in application); 1930

was responsible for and made land valuation of nine canal systems in Kern County serving approx. 250,000 acres, studied crops, soil conditions, topography, pumping costs and water-table fluctuations, calculated 5-year inflow and outflow for underground area (approx. 150 000 acres), additional drains on water available, etc., studied suggested schemes for augmenting underground water supply by dumping excess waters, inspected over 700 miles of canals, etc., gathered data on farming conditions, land values, etc., for pumped water and power cost analysis; 1929 made similar valuation of San Joaquin & Kings River Canal and Irrigating System lands. *TT 11.8: P 11.8: RC 4.*—*TT 19.3: P 19.3: RC 6.* Refers to H. L. Haehl, W. Hall, A. G. Mott, H. A. Noble, W. Stava.

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(16) **SCHANCK, CHARLES ARMSTRONG,** U. S. Coast and Geodetic Survey, Washington, D. C. (Age 28. Born Roberts, Mont.) 1924 B. S., Mont. State Coll. *TT 4: P 4.*—July 1924 to Jan. 1925 and June 1925 to date with U. S. Coast and Geodetic Survey, successively as Lightkeeper and Recorder, Jun. Engr. (1 2/5 years), Aid (3 years) and (since Feb. 1930) Jun. Hydrographic and Geodetic Engr., on office work, triangulation, leveling, combined operations and astronomy, on training ships, etc., acting as Instructor on first-order leveling, Asst., Chf. of Party on levels, triangulation, reconnaissance, etc. *TT 7: SP 0.4: P 4.6: RC 3.5: D 1.7.*—*TT 11: SP 0.4: P 10.6: RC 3.5: D 1.7.* Refers to H. G. Avers, F. S. Borden, W. Bowie, C. L. Garner, H. W. Hemple, H. S. Rappleye, H. A. Seran.

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(1) **SCHMIDT, PAUL HENNING,** 431 Conrad Road, Englewood, N. J. (Age 43. Born Copenhagen, Denmark. Prior to 1912 (4½ years) student in Civ. Eng., Univ. of Copenhagen, Denmark. *TT 2: P 2.*—Oct. 1913 to June 1915 Transitman and Estimator, Office of Res. Engr., Canadian Govt. Rys., on surveys, drafting, estimates, construction supervision of track realignment, grade, culvert and bridge construction. *TT 0.8: SP 0.8.*—Aug. to Dec. 1915 and Jan. 1917 to May 1918 with Detroit (Mich.) Edison Co., first on appraisal of industrial buildings, power and central-heating plants, and after Jan. 1917 Estimator on construction cost analysis, specifications for construction of electric power and central heating plants. *TT 1.5: SP 0.2: P 1.3.*—Jan. 1916 to Jan. 1917 Transitman, Michigan Central Ry. *TT 0.5: SP 0.5.*—May 1918 to June 1929 with Sinclair Refining Co., and June 1929 to date with Sinclair Consolidated Oil Coporation, until Sept. 1924 as Estimating Engr., at Chicago, Ill., in charge of estimates for petroleum refineries and terminals, revaluation of refinery property, classification of construction cost of property records, valuation of refinery properties and supervision of cost departments in six refineries under construction; Sept. 1924 to June 1929 supervising Property Dept., New York City, reclassifying and recording construction expenditures for refinery, terminal and marketing facilities, values, depreciation studies, etc.; since June 1929 Insurance and Valuation Engr., New York City, making periodic inspections of refineries, terminals, market facilities, reports on condition, recommendations on fire-prevention and fire-fighting equipment, valuation of properties, study of construction cost and depreciation, perpetual records, etc. *TT 13.8: P 13.8: RC 7.5.*—*TT 18.6: SP 1.5: P 17.1: RC 7.5.* Refers to E. T. Almy, Jr., J. S. Hess, J. Lucas, J. B. Stein, K. K. Wyatt.

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(3) **SUMMERS, WAYNE,** 397 State St., Albany, N. Y. (Age 39. Born in Wells County, Ind.) (1916 B. S. in C. E., Purdue Univ. *TT 4: P 4.*—June 1916 to Sept. 1917 Asst., Engr. Corps, Pennsylvania Ry., being Chf. of Survey Party on realignment and spiraling of curves, made report, etc., later had charge of bridge repairs and culvert construction, Ft. Wayne Div., designed a reinforced concrete box culvert, laid out industrial sidings, etc. *TT 1.3: P 1.3: RC 0.5: D 0.5.*—Sept. 1917 to July 1919 Master Engr., 309th Engrs., and 2d Lieut., 603d Engrs., U. S. Army, 10 months in France, pontoon bridge train and road construction and repair. *TT 1.6: SP 0.2: P 1.4: RC 1.4.*—July 1919 to Dec. 1921 with Indiana State Highway Comm. as Inspector in charge of concrete highway construction, also on design and drafting, later Dist. Supt., Div. of Maintenance, in charge of maintenance and reconstruction, involving ditching and gravel and stone surfacing. *TT 2.1: SP 0.2: P 1.9: RC 1.9.*—Jan. 1922 to Feb. 1926 Sales Engr., J. D. Adams & Co., Indianapolis, Ind., work involved demonstrating and teaching economical use of road grader. *TT 4.: P 4.*—Feb. 1926 to May 1931 with W. R. Grace & Co., New York City, as Sales Engr. and Mgr. of Road Machinery and Materials Div., International Machinery Co. in Brazil, on similar work, also responsible for selection of power excavators, concrete mixers and placers, tractors, hoists and other equipment for a 57-mile, 9-ft. water conduit; also taught engineers in charge how to use equipment; had direct charge of office and selling

organization. *TT 5.1: P 5.1: RC 4.1.*—May 1931 to date Sales Engr., American Bitumuls Co., Baltimore, Md., in charge in eastern New York State. *TT 1: P 1.*—*TT 19.1: SP 0.4: P 18.7: RC 7.9: D 0.5.* Refers to P. L. Boneysteele, P. L. Fahrney, A. H. Hinkle, C. L. McKesson, F. T. Sheets, F. T. da S. Telles, J. H. Thomas.

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(1) WALSH, WALTER VINCENT, 18 East Forty-first St., New York City. (Age 37. Born New York City.) Student, Coll. of City of New York (1913–1915) and Cooper Union Inst. of Technology (1916, nights). *TT 1.5: P 1.5.*—Sept. 1915 to June 1916 Eng. Draftsman with George Hill, Cons. Engr., drafting, layout and design of paper mills and manufacturing plants (under supervision). *TT 0.4: SP 0.4.*—June 1916 to Feb. 1917 Jun. Asst. Engr., Public Service Comm.. New York State, appraising gas plants, buildings and equipment, listing equipment and determining values of buildings and plants of Newtown Gas Co. *TT 0.4: SP 0.4.*—Feb. to Oct. 1917 and April 1920 to May 1922 Engr., Henry Hope & Sons, Steel Window Mfrs., New York City, in charge of drafting room, design and field erection of large steel windows with special mechanical operating gear for banks and public buildings; had complete charge of preparing shop drawings for their manufacture in England. *TT 2.7: P 2.7: RC 1.7: D 1.*—Oct. 1917 to April 1920 Senior Inspector of Airplanes and Airplane Engrs., Inspection Sec., Air Service, U. S. Army, 3 months taking course at Curtis Airplane Co. plant, Buffalo, N. Y., 6 months Inspector at various plants, and later Senior Inspector in charge of inspection at factories in Plainfield, Elizabeth and New Brunswick, N. J. *TT 2.2: SP 0.2: P 2: RC 2.*—May 1922 to date Vice-Pres. and Treas. and member of firm, John H. Eisele Co., Inc., New York City, erecting banks, factories, schools and churches ($100 000 to $1 500 000 each) from architects' and engineers' plans and specifications, supervising field superintendents, attending conferences with owners, architects and engineers and coordinating work of sub-contractors. *TT 10: P 10: RC 10: D 1.*—*TT 17.2: SP 1: P 16.2: RC 13.7: D 2.* Refers to J. B. Clermont, C. H. Higginson, T. H. Irving, Y. M. Karekin, C. B. Spencer.

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(3) WISEWELL, FRANCIS HENRY, 16 Cottage St., Dansville, N. Y. (Age 48. Born Phelps, N. Y.) 1901–1903 student, Pratt Inst.—Sept. 1903 to Aug. 1906 Draftsman and Engr., Continental Car and Equipment Co., New York City, drafting on railroad and contractors equipment, inspecting railroad cars in plants of American Car and Foundry Co.; Engr. in charge. *TT 2: SP 1: P 1: RC 1: D 1.*—Jan. to Oct. 1907 Asst. Engr., Westinghouse, Church, Kerr & Co., New York City, estimating costs on power and industrial plants, railroad shops and terminals. *TT 0.4: SP 0.4.*—April 1908 to May 1911 Salesman, International Heater Co., and Vice-Pres., Mohawk Valley Heating Co., Utica, N. Y., designing, estimating costs and installing heating and ventilating plants and piping systems. *TT 2.1: SP 1.1: P 1: RC 1: D 1.* —April 1912 to June 1913 Draftsman, Baldwin Locomotive Works, Philadelphia, Pa., drafting on general locomotive work. *TT 0.6: SP 0.6.*—Oct. 1913 to April 1917 Draftsman and Chf. Draftsman, and Nov. 1919 to Dec. 1924 Designer for Gifford Wood Co., Hudson, N. Y., on design of elevating and conveying machinery installations, steel and timber structures, coaling facilities for power plants and railroads, etc. *TT 4.3: SP 1.2: P 3.1: RC 1: D 3.1.*—May 1917 to July 1919 1st Lieut. and Capt., Corps of Engrs., U. S. Army, in United States, France and England, on construction of camps, hospitals, warehouses, etc., purchase, storage and liquidation of engineer and contractor equipment. *TT 2.1: SP 0.1: P 2: RC 1.*—April 1925 to July 1927 Asst. to Plant Engr., United States Gypsum Co., Oakfield, N. Y., on design of steel mill buildings, rock bunkers and general alteration of gypsum plant. *TT 2.3: P 2.3: RC 2.3: D 2.3.*—April 1928 to July 1931 Draftsman, Foster-Wheeler Corporation, Dansville, N. Y., designing steel structures. *TT 2.3: P 2.3.*—*TT 16.1: SP 4.4: P 11.7: RC 6.3: D 7.4.* Refers to A. S. Ackerman, E. V. R. Payne, E. H. Sargent, M. M. Upson, H. S. Wilgus, G. C. Wright.

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(9) WOLFE, THEODORE, 9811 Parkgate Ave., Cleveland, Ohio. (Age 28. Born Baltimore, Md.) 1924 B. C. E., Ohio State Univ. *TT 4: P 4.*—At present (evenings) graduate student, Case School of Applied Science.—1924 to 1926 Road Constr. Engr., Ohio Contr. Co., and (winters) Structural Draftsman, Industrial Brown Hoist Corporation, Cleveland, Ohio. *TT 1.5: SP 0.5: P 1: RC 1: D 0.3.*—1926 to 1927 with Chas. S. Schneider, Morris & Weinberg, and Lyman S. Walker, Archts., Cleveland, on engineering design and superintendence of buildings, theatres, etc. *TT 2: P 2: RC 2: D 2.*—March to Dec. 1927 with Carson G. French, Structural Engr., Cleveland, on steel and concrete inspection, being responsible for inspection of structural work on warehouse buildings and railroad

structures. *TT 0.8: P 0.8: RC 0.8.*—March 1928 to Jan. 1930 with Nimmons, Carr & Wright, Archts., Chicago, Ill., as Supt. of Constr. on reinforced concrete commercial and industrial buildings (architectural and structural), handling design changes in field. *TT 1.8: P 1.8: RC 1.8: D 1.5.*—June 1930 to June 1931 Structural Designer and Squad Capt., Arthur G. McKee Co., Cleveland, designing steel and reinforced concrete structures for Russian Govt., *Magnitostroy* steel plant. *TT 1: P 1: RC 0.7: D 1.*—June 1931 to date Structural Designer and Draftsman, Van Dorn Iron Works, Cleveland, on structural steel design and detail. *TT 0.9: P 0.9.*—*TT 12: SP 0.5: P 11.5: RC 6.3: D 4.8.* Refers to C. G. French, B. R. Magee, C. T. Morris, F. L. Plummer, C. C. Wright.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

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(1) ALVAREZ y de URRUTIA, ARMANDO MANUEL, Assoc. M., P. O. Box 1399, Lima, Peru. (Elected Junior Jan. 19, 1920; Assoc. M. May 19, 1924.) (Age 35. Born Remedios, Cuba.) 1918 B. S. in C. E., and 1920 C. E., Villanova (Pa.) Coll. 1920 C. E., Havana Univ. *TT 4: P 4.*—Summer 1916 Rodman, Philadelphia & Reading R. R., inspecting concrete work, etc. on a bridge.—Summer 1917 Concrete Foreman with Seeds & Derham on construction of 3-span concrete arch bridge near Allentown, Pa., for P. & R. R. R., and with Babcock & Wilcox, on foundations for new machine shop at Bayonne, N. J.—June to July 1918 Rodman, Pennsylvania R. R., being Draftsman, Harrisburg Div.—Aug. 1918 to Dec. 1920 Eng. Draftsman and Field Engr., Snare & Triest Co., Gen. Contrs., on concrete detailing, estimating, location of bridges, piers, wharves, house construction, etc. *TT 1.2: SP 1.2.*—Jan. 1921 to date with Frederick Snare Corporation, Contr. Engrs., until April 1928 as Asst. Engr., Havana office, designing and construction engineering, superintendence and inspection; designing, estimating and inspecting, during construction, concrete and steel structures for bridges, buildings, piers, warehouses and residences; investigation for construction of engineering structures, etc., and since May 1928 Mgr. and Chf. Engr., Lima (Peru) office, on final design and construction of new port works, Callao, Peru (over $8 000 000); from Jan. 1921 to April 1928 also private work as Designer for several firms and appraisal in legal cases. *TT 11.3: P 11.3: RC 11.3: D 11.3.*—*TT 16.5: SP 1.2: P 15.3: RC 11.3: D 11.3.* Refers to E. J. Chibas, M. Font, F. J. Gaston, F. J. Litter, G. P. Seeley, Jr., E. S. Skillin, M. Villa Rivera, R. S. Webster.

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(8) GAYTON, LORAN DeLANCY, Assoc. M., 402 City Hall, Chicago, Ill. (Elected April 25, 1921.) (Age 51. Born Boston, Mass.) 1910 to 1912 special student, Univ. of Illinois.—1904 and 1907 with O. E. Strehlow, Chicago, Ill., as Foreman in charge of placing reinforcing steel in construction of concrete building, and (1907) Asst. to Engr. in charge of construction of dam on Illinois River, Dresden Heights, Ill. *TT 1: P 1: RC 1.*—1904 to 1906 with Union Station Co., Chicago, as Asst. to Engr. in charge of construction of Jefferson St. Bridge (three 110-ft. spans), South Bend, Ind. *TT 2: P 2: RC 2.*—1906 to 1907 in charge of concrete construction for track elevation, Grand Trunk Ry., Chicago. *TT 0.5: P 0.5: RC 0.5.*—1908 to 1910 with G. E. Kahn, Milwaukee, Wis., as Engr. in charge of placing pile and mat foundations for two buildings (6 months) and of construction of Grand Ave. Viaduct (eight 145-ft. spans, approx. $500 000) (2 years). *TT 2.5: P 2.5: RC 2.5.*—Summer 1911 Asst. Engr. in charge of construction of dam at Prairie du Sac, Wis.—Summer 1912 Field Engr. on valuation of Canadian Pacific R. R. at Montreal and Toronto, Canada.—1913 Engr. in charge of construction of Gordon Ferguson Bldg. (concrete), St. Paul, Minn., Mr. Vandanacker, Contr. *TT 0.5: P 0.5: RC 0.5.*—1914 to date with City of Chicago, until 1927 in charge of design of cribs, tunnels, pumping stations, etc., for water-supply system, being Designing Engr. (4 years), Asst. Engr. of Water-Works Design (4 years), and Engr. of Water-Works Design (4 years), 1927 to 1931 City Engr., and since 1931 Asst. City Engr. *TT 17: P 17: RC 17: D 8.*—*TT 23.5: P 23.5: RC 23.5: D 8.* Refers to W. J. Cahill, T. L. Condron, W. W. DeBerard, A. J. Hammond, R. W. Putnam, A. F. Reichmann, O. E. Strehlow.

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(10) HALL, BENJAMIN MORTIMER, Jr., Assoc. M., 701 Peters Bldg., Atlanta, Ga. (Elected May 15, 1917.) (Age 40. Born Atlanta, Ga.) 1912 B. S. in Mech. Eng., Ga. School Tech. *TT 4: P 4.*—Summers Field Asst., U. S. Geological Survey (1907),

and with Porto Rico Irrigation Service (1909), gauging streams, and with Amicalola Marble Co., Ball Ground, Ga., estimating stone work (1911).——July 1912 to April 1913 Instrumentman and Draftsman, Hall Bros., Civ. and Min. Engrs., Atlanta, Ga., on surveys, etc. *TT 0.4: SP 0.4.*——May to Sept. 1913 Asst. Hydr. Engr., Florida Everglades Eng. Comm., on hydrographic work, in charge of measurements of rainfall, evaporation and out-flow, Lake Okeechobee. *TT 0.4: P 0.4: RC 0.2.*——Oct. 1913 to April 1914 Jun. Engr., U. S. Geological Survey (Water Resources), stream gauging in southeastern states also private work with Hall Bros., being Asst. Engr. on surveys and design of water powers in Georgia and North Carolina. *TT 0.4: SP 0.2: P 0.2: D 0.2.*——May 1914 to July 1915 Jun. Engr., U. S. Reclamation Service in Utah, until May 1915 as Hydrographer, Strawberry Valley Project, in charge of and wrote reports on steam gauging, and reports on inflow and evaporation, Strawberry Reservoir, and after May 1915 Concrete Inspector on flumes, siphons, turnouts and canal lining. *TT 1.3: P 1.3: RC 1.*——Aug. 1915 to Jan. 1917 member of firm, Hall Bros., and Jan. to April 1917 of firm B. M. Hall & Sons, hydraulic and mining work, etc., being Asst. on design and supervised construction of earth dam, water supply and sewage-disposal system, Ingleside Country Club, Atlanta, rock-filled dam, canal and power house, Chestatee River, Ga., Asst. on water-supply and water-power reports, investigations of iron, granite, marble and barytes properties, and report on 50 000-h-p. development, Chattahoochee River (for U. S. Nitrate Plants). *TT 1.7: P 1.7: RC 0.5: D 0.5.*——May 1917 to June 1919 with U. S. Army, until Aug. 1917 at First Officers' Training Camp, Aug. 1917 to Nov. 1918 First Lieut., and Nov. 1918 to June 1919 Capt. Engrs., A. E. F., 26th Engrs. (Water Supply), with troops, on design and construction of water supply systems, commanded Co. B, St. Mihiel; Oct. to Dec. 1918 Asst. Water Supply Officer, 2d Army, and Jan. to April 1919 Acting Officer in charge, Water Supply Sec., Office of Chf. Engr., A. E. F. *TT 2.1: P 2.1: RC 2.1: D 0.5.*——July 1919 to date member of firm, B. M. Hall & Sons, on design and construction of hydro-electric plants at Bainbridge, Ball Ground, and Maysville, Ga., water supply, dams, etc., Berry Schools, Rome, Tate, Jasper and Warm Springs, Ga., and Piedmont, Ala., mine investigations gold, barytes, manganese and ocher, surveys, design, estimates and reports on water powers and water-supply systems, valuations of hydro-electric plants, marble quarries, manganese mines and undeveloped water powers; flood control investigations, including large combined flood-control and water-power dam on Chattahoochee River, West Point, Ga., investigations and expert testimony, flood damage, condemnation, mining and water power valuation cases. *TT 12.8: P 12.8: RC 12.8: D 6.*——*TT 23.1: SP 0.6: P 22.5: RC 16.6: D 7.2.* Refers to C. G. Adsit, J. B. Gordon, N. C. Grover, M. R. Hall, W. E. Hall, J. B. Hawley, J. H. Johnston, L. A. Kibbe, M. O. Leighton, F. H. McDonald, A. H. Pratt, T. Saville, F. C. Snow, C. C. Whitaker.

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(1) KAESTNER, ALBERT CARL, Assoc. M., 2 Lafayette st., New York City. (Age 43. Born Newton, Kans.) (Elected Jun., Jan. 2, 1912; Assoc. M., May 31, 1916.) Prof. Engr., New York State.——1910 C. E., Columbia Univ. *TT 4: P 4.*——Summer 1906 Timekeeper, Columbian Reinforced Concrete Co.——1907 Chainman, June 1910 to Feb. 1911 Transitman, being Acting Chf. of Party on construction, laying out bridge piers and abutments, preparing estimates, etc., Oct. 1911 to Nov. 1912 Chf. of Party and Draftsman, on valuation, and Jan. to March 1914 Asst. Engr., on revaluation, all for Lehigh Valley R. R. *TT 2.1: P 2.1: RC 1.3.*——Feb. to May 1911 Asst. Engr., Bureau of Bldgs., New York City, being Special Inspector. *TT 0.3: P 0.3: RC 0.3.*——May to Oct. 1911 Chf. of Party, New York Central R. R., on valuation. *TT 0.4: P 0.4: RC 0.4.*——Nov. 1912 to Jan. 1914 Asst. Road Master and Chf. Draftsman, Pittsburgh, Shawmut & Northern R. R. *TT 1.1: P 1.1: RC 1.1: D 0.7.*——March 1914 to Jan. 1918 Asst. to Chf. Engr., U. S. Realty & Improvement Co., on New York City subway construction contracts (about $12 000 000). *TT 3.6: P 3.6: RC 3.6: D 1.*——Jan. 1918 to July 1921 Engr., W. S. Kinnear & Co., Cons. Engrs., preparing reports on various railroad projects, industrial investigation and general consulting work. *TT 3.4: P 3.4: RC 3.4: D 1.5.*——July 1921 to April 1925 Secy.-Treas., Farr Hydraulic Systems, Inc., designing, selling and supervising installation of gasoline and oil-handling systems. *TT 3.7: P 3.7: RC 3.7: D 3.7.*——April 1925 to date Organizer, Secy.-Treas., Director and acting as Gen Mgr., Aqua Systems, Inc., which owns and controls patents on hydraulic gasoline and oil handling systems, designs, manufacturers and installs these systems in industrial plants, airports, etc.; since March 1928 also Secy.-Treas. and Director, Farr Hydraulic Systems, Inc. *TT 7: P 7: RC 7: D 7.*——*TT 25.6: P 25.6: RC 20.8: D 13.9.* Refers to G. E. Beggs, T. C. Desmond, W. S. Kinnear, R. P. Miller, A. V. Sielke, C. B. Spencer, C. H. Stengel.

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(12) **MOREELL, BEN,** Assoc. M., Puget Sound Navy Yard, Bremerton, Wash. (Elected Aug. 4, 1924.) (Age 39. Born Salt Lake City, Utah.) 1913 B. S. in C. E., Washington Univ., St. Louis, Mo. *TT 4: P 4.*—June 1913 to May 1917 with Dept. of Sewers, St. Louis, Mo., 1½ years as Designing Engr. and 2½ years as Res. Engr. on construction projects. *TT 3.8: SP 0.1: P 3.7: RC 2.5: D 1.5.*—May 1917 to date with Corps of Civ. Engrs., U. S. Navy, from Aug. 1917 to Jan. 1918 being Asst. to Public Works Officer, New York Navy Yard; Jan. 1918-May 1919 Public Works Officer, U. S. Naval Base No. 13, Ponta Delgada, San Miguel, Azores; June 1919 to Sept. 1920 Superv. Engr., U. S. Navy Destroyer Plant, Squantum, Mass., and Civ. Engr. Member of Plant Board for shipbuilding facilities on East Coast; Sept. 1920–Oct. 1924 Executive Officer to Engr. in Chf., Dept. of Public Works, Haiti; Oct. 1924–May 1926 Asst. to Public Works Officer, Norfolk Navy Yard and (about 3 months) Public Works Officer; June 1926 to June 1930 Prin. Asst. to Design Mgr., Bureau of Yards and Docks, Navy Dept.; since June 1930 Public Works Officer, 13th Naval Dist. and Navy Yard, Puget Sound, Wash. *TT 15: P 15: RC 15: D 4.*—*TT 22.8: SP 0.1: P 22.7: RC 17.5: D 5.5.* Refers to W. W. Horner, G. A. McKay, A. L. Parsons, N. M. Smith, E. O. Sweetser, J. L. Van Ornum, D. C. Webb.

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(9) **NEWSOM, CLIFFORD COOK,** Assoc. M., 1108 East Main St., Crawfordsville, Ind. (Elected Aug. 27, 1928.) (Age 50. Born Elizabethtown, Ind.) Sept. 1900 to June 1902 and Feb. 1904 to Jan. 1905 student, Coll. of Civ. Eng., Purdue Univ. *TT 1.5: P 1.5.*—June 1902 to March 1903 Rodman, Cleveland, Cincinnati, Chicago & St. Louis Ry. *TT 0.4: SP 0.4.*—April 1905 to April 1906 Draftsman and Instrumentman, Chicago & North Western Ry. *TT 0.5: SP 0.5.*—April 1906 to March 1907 on masonry design, Louisville & Nashville R. R., Bridge Dept. *TT 0.9: P 0.9: D 0.5.*—March 1907 to April 1910 and May 1911 to June 1915 private work and county road surveying and contracting, including (March 1914 to April 1915) resurveys and plans for electric line (70 miles) for Interstate Public Service Comm.. Columbus, Ind. *TT 1.7: P 1.7: RC 1.7: D 0.7.*—April 1910 to May 1911 Asst. Engr., M. of W., Illinois Traction Co., in charge of maintenance and construction, including surveys, plans, building of road bed and bridges. *TT 1.1: P 1.1: RC 1.1.*—June 1915 to May 1916 Jun. Engr., Div. of Valuation, Interstate Commerce Comm. *TT 0.5: SP 0.5.*—May 1916 to April 1917 in charge of locating party, Pennsylvania R. R., made surveys for freight yards south of Logansport, Ind., etc. *TT 0.9: P 0.9: RC 0.9.*—April 1917 to March 1920 Asst. Engr., Valuation Dept., Lake Erie & Western R. R., in charge of valuation surveys and plans for railroad properties, except structures. *TT 3: P 3: RC 3.*—March 1920 to date with Indiana State Highway Comm., until April 1922 as Sub-Dist. Supt., and since then Dist. Engr. in direct charge of maintenance and minor betterment work in Crawfordsville Dist. (about 1 250 miles of state roads); has supervised maintenance (approx. $4 000 000) and betterments ($2 000 000), such as building small bridges and constructing approx. 100 miles of bituminous roads. *TT 11: SP 1: P 10: RC 10: D 5.*—*TT 21.5: SP 2.4: P 19.1: RC 16.7: D 6.2.* Refers to C. H. Apple, A. H. Hinkle, F. Kellam, H. S. Marshall, A. P. Melton, C. P. Owens, W. J. Titus.

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(9) **PARKS, WARREN WRIGHT,** Assoc. M., 3800 Plainville Road, Mariemont, Cincinnati, Ohio. (Elected Dec. 4, 1922.) (Age 36. Born Russell, Mass.) 1919 B. S. in C. E., Worcester Pol. Inst. *TT 4: P 4.*—July–Dec. 1917 with Durkee, White & Towne, Engrs., Springfield, Mass., as Rodman and Transitman on construction of Camp Devens and model town at Bristol, Pa., sewers, roads, underground steam lines, building layouts, etc., being responsible for field party, and Jan. 1918–Feb. 1919 with Merchant Shipbuilding Corporation, on same work, being Chf. of Party, responsible to Div. Engr. *TT 1.3: SP 0.2: P 1.1.*—April 1919 to April 1920 Instrumentman, Bridge Dept., Boston & Albany R. R., in charge of field party, being responsible for lines and grades on construction of 9-span main-line viaduct at Westfield, Mass., surveys of other structures, etc. *TT 1: P 1.*—April 1920 to April 1923 with Fay, Spofford & Thorndike, Boston, Mass., as Asst. Engr. in charge, being responsible to Res. Engr. for field engineering on construction of Hampden County Memorial Bridge and approaches ($4 000 000). *TT 3: P 3: RC 3.*—April 1923 to date Res. Engr. in charge of construction of Mariemont (model town, 250 acres, $10 000 000), first year responsible to Fay, Spofford & Thorndike, then to The Mariemont Co. (Promoters) and since Jan. 1932 to The Thomas J. Emery Memorial; work includes responsibility for layout, inspection, supervision, and (last 5 years) for design. *TT 9: P 9: RC 9: D 5.*—*TT 18.3: SP 0.2: P 18.1: RC 12: D 5.* Refers to F. H. Fay, A. W. French, E. D. Gilman, H. D. Loring, J. E. Root.

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(1) **STAUFFER, ISAAC YOST**, Assoc. M., P. O. Box 404, Yokohama, Japan. (Elected Junior May 31, 1916; Assoc. M. May 12, 1919.) (Age 45. Born Boyertown, Pa.) 1909 C. E., Princeton Univ. *TT 4: P 4.*—1909 to 1914 with Riter Conley Mfg. Co., 3 years as Field Engr. on erection of steel penstocks at Necaxa, Mexico, oil tanks at Pittsburgh and at Tampico, Mexico, and gas holder at St. Louis, and 1½ years Engr. in charge on erection of steel penstocks at Fremont, Ohio, and Altmar, N. Y. *TT 3.3: SP 1.5: P 1.8.*—1914 to date with Standard Oil Co. of New York, until 1915 as Draftsman, 1915 to 1918 Engr. of Constr. in Straits Settlements and Dutch East Indies, erected oil-storage plant at Penang ($115 000), also warehouses on marshy ground; 1918 to 1924 Engr. in charge of construction of bulk oil-storage plants at Batavia ($360 000) and Singapore ($500 000), and in general charge of all construction; 1924 to 1927 Engr., assisting in reconstruction of oil-storage plants in Yokohama, and enlargement and maintenance of oil-storage plants at Osaka, Icozaki and Nagasaki; since 1927 Engr. in charge of all construction and maintenance work in Japan and Korea, constructed new godowns, storage tanks, reinforced concrete piers and extended existing facilities. *TT 17.2: SP 0.2: P 17: RC 15: D 7.*—*TT 24.5: SP 1.7: P 22.8: RC 15: D 7.* Refers to D. W. Bliem, K. A. Enz, R. F. Moss, J. A. Shaw, E. A. Silagi, W. W. Stevens.

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(13) **WARREN, DONALD REXFORD**, Assoc. M., 1416 Fifty-second St., Sacramento, Cal. (Elected July 16, 1928.) (Age 35. Born Dayton, Nev.) 1917 to 1920 student, Coll. of Civ. Eng., Univ. of Nev. *TT 2: P 2.*—1923 Diploma in Civ. Eng., International Correspondence School.—June 1920 to Feb. 1921 Draftsman, American Bridge Co., Gary, Ind., detailing structural steel drawings. *TT 0.7: P 0.7.*—March 1921 to Jan. 1922 Engr., Redmond Page Co., on construction of Topaz Reservoir for Walker River Irrigation Dist., tunnel with reinforced concrete lining, reservoir control structures, conduit, canals and roads. *TT 0.7: P 0.7.*—Jan. 1922 to Nov. 1923 Prin. Asst. Engr., Walker River Irrigation Dist., Yerington, Nev., drafting, run-off and return-flow studies, Chf. of Party on surveys; Res. Engr. on location and design of Bridgeport Valley storage works, earth-fill dam, siphon spillway, conduit, reservoir controls, highway, etc. *TT 1.8: P 1.8: RC 0.8: D 0.5.*—Nov. 1923 to Feb. 1924 Engr. with H. E. Chesebro, Lewiston, Cal., investigation and report on hydro-electric development on Bald Mt. Creek, Ore., surveys, design, estimates, etc. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—Feb. 1924 to Nov. 1928 Engr. with The Foundation Co., being Constr. Engr., building Pacific Portland Cement Plant at Redwood City, Cal., pile foundations, reinforced concrete tanks, caissons, buildings, etc., structural steel bridge and craneway, wharfs and installation of machinery; Field Engr. on construction of Big Meadows Dam (hydraulic fill), spillway, conduit, controls, etc. forming Lake Almanor Reservoir (1 300 000 acre ft.), structural steel bridge and highway (17 miles); Engr. of Purchases on building of Pacific Goodrich Tire and Rubber Co.'s factory in Los Angeles, Engr. on construction of deep water piers for San Mateo-Alameda Bridge across San Francisco Bay, design of construction plant, driving piles, coffer dams, caissons and constructing piers. *TT 4.8: P 4.8: RC 3.4: D 1.6.*—Nov. 1928 to Jan. 1929 Office Engr. with R. M. Frandsen, San Francisco, on structural steel plans. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—Jan. to Sept. 1929 Engr., City of San Francisco, on designs for Hetch Hetchy Water Supply, structural steel, reinforced concrete structures and earth-fill dam. *TT 0.8: P 0.8: RC 0.8: D 0.8.*—Sept. 1929 to Sept. 1931 Associate Hydr. Engr., Dept. of Public Works, Div. of Water Resources, State of California, salinity investigation, stream and tidal flow studies of Upper San Francisco Bay and lower channels of the Sacramento and San Joaquin Rivers. *TT 2: P 2: RC 2.*—Sept. 1931 to Jan. 1932 Prin. Asst. Engr., Sacramento Municipal Utility Dist., on designs and estimates, rockfill, arch and buttress dams, tunnels, pipe lines, roads, bridge, also hydro-electric studies and financial analyses. *TT 0.3: P 0.3: RC 0.3: D 0.3.*—Jan. 1932 to date Engr., County of Sacramento, on design Fair Oaks Boulevard Bridge over American River, structural steel, reinforced concrete, etc. *TT 0.4: P 0.4: RC 0.4: D 0.4.*—*TT 13.9: P 13.9: RC 8.1: D 4.* Refers to J. A. Beemer, H. P. Boardman, D. Butler, A. Givan, J. W. Gross, E. Hyatt, C. M. Mardel, R. Matthew, W. A. Perkins, L. W. Stocker.

FROM THE GRADE OF JUNIOR

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(9) **ALLEN, GLENN HAROLD**, Jun., 516 South Runnymede Ave., Evansville, Ind. (Elected Oct. 2, 1922.) (Age 32. Born Milan, Ill.) 1922 C. E., Univ. of Cin. *TT 4: P 4.*—July 1922 to April 1923 Draftsman, American Bridge Co., Pencoyd, Pa. *TT 0.8: P 0.8.*—April 1923 to date with International Steel Co., Evansville, Ind. until Jan. 1924

as Draftsman, then Chf. Designing Engr., since Jan. 1922 also acting as Gen. Field Supt. in charge of erection; designed steel work and interior steel framing for motor and fan supports, etc., for Graham-Paige body-plant at Evansville, Ind., steel work for Graham Bros. body-plant, Wayne, Mich., for 100-ft. Fink Truss supporting two 50-ft. cranes for Mead Johnson Terminal, Evansville; designed and patented small bascule bridge; supervised design of machinery for two 220-ft. swing spans, designed three 20-ton derrick travelers for bridge erection, a 20-ton stiff-leg derrick for handling steel, falsework and erection equipment for bridge across Tennessee River, cooperating with F. M. Masters, a building (100 by 600 ft.), including foundations for Company's fabricating shop, also cooperated with architects and engineers on various buildings, churches, etc.; since Jan. 1932 superintending construction of two bridges, one at Sterlington, La., and one at Harrisonburg, La. *TT 9: P 9: RC 8.3: D 8.3.*—*TT 13.8: P 13.8: RC 8.3: D 8.3.* Refers to H. R. Creal, E. L. Erickson, N. E. Lant, F. M. Masters, C. R. Tiebout.

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(1) **ANDREWS, ERIC ALEXIS,** Jun., 85 Rockledge Ave., White Plains, N. Y. (Elected Dec. 3, 1928.) (Age 29. Born Bordeaux, France.) 1927 C. E., Cornell Univ. *TT 4: P 4.* —June 1927 to Jan. 1928 Jun. Asst. Engr., New York State Highway Dept., being Inspector on construction of concrete pavements and at asphalt plant and Asst. to Engr., in charge of grade-elimination project over Long Island R.R. *TT 0.3: SP 0.3.*—Feb. 1928 Transitman, Westchester County San. Comm., White Plains, N. Y.—March 1928 to Jan. 1929 Asst. Engr. with James H. Fuertes, Cons. Engr., New York City, being Transitman and Chf. of Party on sewer survey, Linden, N. J., Draftsman and Computer in office, Designer of storm-water drains and sanitary sewers, Estimator, etc. *TT 0.7: SP 0.3: P 0.4: D 0.4.* —Feb. 1929 to date Engr. of Sewers, Dept. of Public Works, White Plains, in responsible charge of design and construction of sanitary sewerage and storm-water drainage projects; designed and supervised construction of sewers and drains (25 miles, $700 000); work included charge of contract plans, specifications and estimates and of field parties on preliminary surveys; made designs, studies and calculations and supervised field parties giving line and grade on construction, directed inspectors, signed estimates, etc. *TT 3.2: P 3.2: RC 3.2: D 3,2.*—*TT 8.2: SP 0.6: P 7.6: RC 3.2: D 3.6.* Refers to T. A. Avery, F. A. Barnes, R. H. Gould, F. J. Laverty, J. E. Perry.

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(1) **ARANIBAR, ERNESTO,** Jun., Casilla 557, La Paz, Bolivia. (Elected Feb. 25, 1924.) (Age 32. Born Cochabamba, Bolivia.) 1923 C. E., Rens. Pol. Inst. *TT 4: P 4.*— Summer 1922 Inspector, New York State Highway Comm.—June 1923 to date with The Foundation Co., until Sept. 1924 as Asst. to Field Engr., building foundations for Telephone Bldg., New York City; Sept. to Oct. 1924 in charge of engineering office for construction of coke and by-products plant, Troy, N. Y.; Oct. 1924 to Aug. 1925 sent to Govt. of Peru as Cons. Engr., having charge of construction of a cement plant, tunnels for water supply of Lima and reports on railroads and water supplies; Aug. 1925 to Nov. 1931 Asst. Mgr., Bolivian Office, designed and built two hydro-electric plants (200 kw. and 300 kw.), designed construction equipment, and had charge of construction of a cement plant, a cotton mill and a 1 800-kw. hydro-electric plant, including a dam, and (after May 1930) had direct charge of sanitation work of City of Oruro, including construction of water-distribution system, storm-water and domestic sewerage systems and 77 blocks of asphaltic concrete on cement concrete base (total about $1 800 000); since Nov. 1931 Acting Mgr. of Bolivian Office. *TT 8.8: P 8.8: RC 7.5: D. 2.3.*—*TT 12.8: P 12.8: RC 7.5: D 2.3.* Refers to H. J. Deutschbein, J. W. Doty, W. Greenalch, T. R. Lawson,, P. C. Ricketts, T. A. Stiles.

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(1) **BIRKE, HAKAN DANIEL,** Jun., 122 Underhill Ave., Brooklyn, N. Y. (Elected July 14, 1930.) (Age 28. Born Orkelljunga, Sweden.) 1927 C. E., Chalmers Tech. Inst., Sweden. *TT 4: P 4.*—1929 to date graduate student, Polytechnic Inst. of Brooklyn.— Feb. 1928 to date Designer, until March 1929 with Allm Betongbyggnadsbyran, Stockholm, Sweden, on bridges, buildings and cable towers, April 1929 to April 1930 with Gibbs & Hill, Cons. Engrs., New York City, on catenary and signal bridges, transmission towers, foundations, guys and guy anchors for railroad electrification, after Aug. 1929 being in charge of Design Squad; since April 1930 with Ambursen Dam Co., New York City, acting as Designing Engr. on designs, plans and estimates for hydro-electric, water supply and irrigation projects. *TT 4.2: P 4.2: RC 2.8: D 4.1.*—*TT 8.2: P 8.2: RC 2.8: D 4.1.* Refers to C. W. Burkland, E. H. Burroughs, Jr., C. V. Davis, J. G. A. Johnson, P. F. Kruse, E. J. Squire, S. W. Stewart.

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(1) BROWN, FREDERICK THEODORE, Jun., 11 Highland Place, Waverly Hills, Little Neck, N. Y. (Elected Dec. 14, 1925.) (Age 30. Born Brooklyn, N. Y.) 1927 B. S. in C. E., Cooper Union Inst. of Tech. (night). *TT 4: P 4.*—Feb. to Aug. 1923 Tracer and Draftsman, Bridge Dept., Long Island R. R.—Aug. to Oct. 1923 Instrumentman, The Transit Comm., New York City, on lines and grades for subway in Flushing.—Oct. 1923 to Aug. 1924 with New York Central R. R., drafting and some minor design.—Aug. 1924 to July 1925 with Murrie & Co., New York City, appraisal work, estimating.—July 1925 to Nov. 1930 Asst. Supt. and Acting Supt. of Constr., Turner Constr. Co., New York City, drafting, estimating and designing, in charge of lines and grades on large structures in New York City, superintending construction of buildings, including heavy industrial buildings and special foundations for office buildings up to 50 stories, Engr. in charge of complicated reinforcement in special concrete structures. *TT 3.5: P 3.5: RC 3.5: D 0.5.*—Nov. 1930 to April 1931 Supt. of Constr., D'Orio Concrete Constr. Co., New York City, on apartment house construction. *TT 0.4: P 0.4: RC 0.4.*—May 1931 to date Constr. Engr. and Cost Appraiser of Bldgs., Title Guarantee & Trust Co., Brooklyn, N. Y., inspecting buildings under construction, rejecting poor construction and recommending improvements for increased structural safety and life of buildings, cost appraisal. *TT 1: P 1: RC 1.*—*TT 8.9: P 8.9: RC 4.9: D 0.5.* Refers to C. F. Giraud, H. G. Hauck, A. B. Heiser, H. C. Paddock, P. Penhune, W. C. Rohdenburg, T. A. Smith.

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(1) DOMAS, JOSEPH JOHN, Jun., 2168 Twenty-fourth St., Long Island City, N. Y. (Elected Oct. 1, 1926.) (Age 27. Born New York City.) 1926 C. E., Brooklyn Pol. Inst. *TT 4: P 4.*—1926 to 1927 graduate student in San. Eng., Columbia Univ.—June 1927 to March 1932 Asst. Engr. with Nicholas S. Hill, Jr., Cons. Engr., New York City, on a 52-in. water-transmission main, filter, water-treatment and sewage-treatment plants, plant valuation, hydraulic studies, etc., in New Jersey and New York (names given in application), also analyses of water and sewage, estimates, reports, etc. *TT 4.7: P 4.7: RC 2.2: D 0.9.*—March 1932 to date Asst. Hydr. Engr., Hackensack (N. J.) Water Co., on design. *TT 0.1: P 0.1: RC 0.1: D 0.1.*—*TT 8.8: P 8.8: RC 2.3: D 1.* Refers to A. N. Aeryns, G. H. Buck, H. H. Chase, E. W. Clarke, H. P. Hammond, N. S. Hill, Jr., N. I. Kass, S. K. Knox, W. P. Seifert, G. F. Wieghardt.

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(16) FITHIAN, WILLIAM ROBBINS, Jun., 302 East Sixty-seventh St., Kansas City, Mo. (Elected June 10, 1929.) (Age 32. Born Bridgeton, N. J.) 1922 B. S., Lafayette Coll. *TT 4: P 4.*—Summer 1920 Asst. to County Engr., Cumberland County, N. J., drafting, highway surveying and construction.—July to Nov. 1922 Supt. for Frank B. Sweeten, Contr., supervising construction of concrete paving. *TT 0.3: P 0.3: RC 0.3.*—April 1923 to date with Missouri Pacific R. R. Co., until Aug. 1929 as Rodman (6 months) and Instrumentman, on maintenance, surveys, plans, estimates and drafting, 6 months in charge of party laying out yard reconstruction in Kansas City, 1 2/5 years handling Asst. Engr.'s work on Northern Kansas Div., and 1½ years in charge of engineering and party on flood-protection work; Aug. to Dec. 1929 Asst. Engr. in charge of and generally supervising flood-protection work on Central Kansas Div.; April to Dec. 1930 Asst. Roadmaster, in charge of work trains, etc., and supervising work; Dec. 1929 to April 1930 and since Dec. 1930 Topographical Instrumentman on maintenance, surveys, plans, estimates and drafting. *TT 7.9: SP 1: P 6.9: RC 4.5.*—*TT 12.2: SP 1: P 11.2: RC 4.8.* Refers to W. J. Burton, H. W. Crawford, E. A. Hadley, A. W. Hefling, A. A. Miller, J. C. Remington, Jr., W. H. Vance, L. Winship.

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(1) HOYT, KENNETH, Jun., 3871 Sedgwick Ave., New York City. (Elected Oct. 14, 1929.) (Age 27. Born Brooklyn, N. Y.) 1927 C. E., Pol. Inst. of Brooklyn. *TT 4: P 4.*—July 1924 to date Engr., William V. Kiehnle Co., New York City, Bldrs. & Engrs., and Gen. Contrs. in building construction; analyzing and estimating costs of engineering (structural) requirements of construction of buildings and support of adjacent structures, design of construction, steel, concrete, masonry, wood-framing, plans and specifications, supervising and checking detail plans of construction, awarding of sub-contracts, etc., laying out and supervising construction and ordering material, being responsible for continuity of construction, also (nearly 5 years) has had partial responsible charge on building projects and full responsible charge on several projects. *TT 4.9: P 4.9: RC 4.9.*—*TT 8.9: P 8.9: RC 4.9.* Refers to H. R. Codwise, A. Fellheimer, H. P. Hammond, T. Heatley, W. A. Kiehnle.

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(1) **LI, SHU-T' IEN**, Jun., 14 Yung Ho Li, Singapore Road, Tientsin, China. (Elected June 7, 1926.) (Age 32. Born Chih Li, China.) 1923 B.S. in C.E., Pei-Yang Univ. 1926 Ph.D. in Civ. Eng., Cornell Univ. *TT 4: P 4.* Summer 1921 Draftsman, Workshop Extension Service, Pei-Yang Univ., Tientsin.—March to April 1922 Draftsman, American Third Asiatic Expedition, Tientsin.—Summer 1922 Topographical Draftsman, Survey Dept., Chihli River Comm.—Sept. 1922 to Feb. 1923 (while student) Research Asst., Testing Materials Laboratory, Pei-Yang Univ.—Sept. 1923 to Sept. 1926 Tsing Hua Fellow, and (1925–1926) also McGraw Fellow, Graduate School, Cornell Univ.; made extensive researches in engineering economics of railroad operation; Summer 1924 and June to July 1925 Structural Detailer, at Elmira Plant, American Bridge Co., and (1925) in Eng. Dept., Post & McCord, Inc., New York City; July to Sept. 1925 Special Asst. Structural Designer, Westchester County Park Comm. *TT 3: P 3: RC 1: D 1.5.*—Sept. 1926 to May 1927 Asst. Engr., Waddell & Hardesty, Cons. Engrs., New York City, designing and checking shop drawings of cantilever and vertical lift bridges. *TT 0.8: P O 8: RC 0.5: D O 8.*—May to Sept. 1927 inspecting railroads, bridge and harbors in Europe, Asia and Africa. *TT 0.3: P 0.3: RC 0.3.*—Sept. 1927 to Jan. 1928 Prof. of Civ. Eng., Feb. to Aug. 1928 Prof. of R.R. and Highway Eng., and Sept. 1931 to date Lecturer in Harbor Eng., Pei-Yang Univ.; also, Sept. 1928 to July 1930 Lecturer of Mechanics and Hydraulics, Hopei Inst. of Technology, Tientsin; April 1929 to July 1931 Tech. Adviser, Dept. of Constr., Liao-ning Prov. Govt., Mukden; Sept. 1929 to date Lecturer in Hydraulics, Coll. of Eng. and Commerce, Tientsin; May 1930 to date Dean, and since Nov. 1930 Director, Research Inst., Tangshan Eng. Coll., Chiao Tung Univ., Tangshan, North China; Oct. to Dec. 1929 Delegate to World Eng. Congress and World Power Conference, Tokyo, representing National Constr. Comm. and Chinese Eng. Soc.; May 1929 to Oct. 1931 Associate Director, Deputy Director and Director, Development Board, and Nov. 1931 to date Chairman, Comm. for Development, Great-Northern-Port, Tientsin; April to Dec. 1931 Cons. Engr., Yung Ting Ho Dike Breach Reconstruction Board, Tientsin; since 1928 also Member, Technical Member, Executive Member, etc. of various Councils, Comms., etc. *TT 4.6: P 4.6: RC 4.6: D 2.3.*—*TT 12.7: P 12.7: RC 6.4: D 4.6.* Refers to F. A. Barnes, A. G. Hayden, E. W. Schoder, L. C. Urquhart, J. A. L. Waddell, T. C. S. Yen.

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(12) **NICHOL, FRANK ELLIOTT**, Jun., 988 Skidmore St., Portland, Ore. (Elected Jan. 17, 1927.) (Age 31. Born Hendrum, Minn.) 1925 B. S. of C. E., and 1926 M. S. of C. E., Univ. of Minn. *TT 4: P 4.*—June to Sept. 1925 Materials Inspector, Minnesota State Highway Dept., *TT 0.1: SP 0.1.*—Oct. 1925 to Aug. 1926 Teaching Fellow and Graduate Student, and Sept. 1926 to June 1927 Instructor in Civ. Eng., Univ. of Minnesota. *TT 0.8: P 0.8.*—July 1927 to Feb. 1929 Engr., Feb. 1929-July 1931 Dist. Engr., and July 1931 to date Sales Engr., Truscon Steel Co., Portland, Ore., and Seattle, Wash., detailing, estimating and designing reinforced concrete and structural steel buildings, and (since July 1931) also selling engineering products for buildings and bridges. *TT 4.8: P 4.8: RC 3.1: D 2.3.*—*TT 9.7: SP 0.1: P 9.6: RC 3.1: D 2.3.* Refers to F. Bass, H. F. Blood, A. S. Cutler, H. M. Hill, G. A. Maney, D. O. Nelson, J. I. Parcel.

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(1) **RAPP, GEORGE MARVIN**, Jun., 243 Grove St., Montclair, N. J. (Elected March 12, 1923.) (Age 32. Born Glens Falls, N. Y.) 1922 C. E., Rens. Pol. Inst. *TT 4: P 4.*—June 1922 to Jan. 1923 Structural Draftsman, Bethlehem Steel Co., Steelton, Pa., detailing and checking shop drawings for large steel office buildings. *TT 0.3: SP 0.3.*—Jan. 1923 to Feb. 1924 Jun. Engr., and Feb. 1924 to May 1926 Asst. Engr., with Delaware River Bridge Joint Comm., Philadelphia, assisting on design of towers, cables, stiffening trusses and approach structures of Philadelphia-Camden Suspension Bridge, including stress analyses, original layout and design in concrete and steel, estimates, checking, laboratory and field tests, calculation of erection stresses and deflections for wire cables and stiffening trusses and determination of erection procedure, assisting in shop steel inspection (two months) and reports. *TT 3.3: P 3.3: RC 2.2: D 2.2.*—May 1926 to date Asst. Engr., The Port of New York Authority, New York City, until Jan. 1928 in charge (under Asst. Engr. of Design) of design of wire cables, steel anchorages and future stiffening trusses of George Washington Bridge, and Jan. 1928 to Oct. 1931 in charge (under Engr. of Design) of checking contractor's drawings and supervising contract work for Contract HRB-5B for wire cables and anchorage steelwork, for same bridge, including checking erection structures and directing erection procedure; also since June 1930 assisting Engr. of Design, on planning and administration of Research and Tests Sec., Port Authority Laboratory, including special laboratory tests and field measurements of strain in bridge

members and tunnel lining and reports, and (since July 1930) in administrative work for purchases, specifications, planning of contract work and miscellaneous design on all Port Authority bridges. *TT 5.9: P 5.9: RC 5.9: D 4.4.*—*TT 13. 5: SP 0.3: P 13.2: RC 8.1: D 6.6.* Refers to O. H. Ammann, M. B. Case, C. E. Chase, A. Dana, L. S. Moisseiff, E. W. Stearns.

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(1) TOPPING, CHARLES HINCHMAN, Jun., 106 West Fifty-sixth St., New York City. (Age 27. Born Yonkers, N. Y.) (Elected Nov. 11, 1929.) Licensed Surveyor.—1929 B. S., Mass. Inst. Tech. *TT 4: P 4.*—March to May 1926 Recorder and Instrumentman with New Mexico State Engr., Santa Fe (1 month), and Instrumentman and Draftsman, American Metals Co., New Mexico, on construction of shops and equipment for Pecos Mines (1 month). *TT 0.1: SP 0.1.*—Aug. 1925 to Oct. 1927 (at intervals) Contr., Santa Fe, New Mexico, designed and built stables, buildings and rifle range for New Mexico National Guard, swimming pools, houses, sewers, irrigation systems and roads, made land surveys, etc. *TT 1: P 1: RC 1: D 1.*—June 1928 to June 1930 with Lago Petroleum Corporation, Maracaibo, Venezuela, three months as Instrumentman, four months First Asst. Constr. Engr. and Chf. of Party on tank farms, railroads, swamp drainage and camps (buildings, roads, sewers, sidewalks, watermains, etc.), two months Draftsman and Designer, on piers, rig foundations and water-front structures and 1¼ years Cost. Engr., in charge of Cost-Keeping and Estimating Dept. *TT 1.8: SP 0.2: P 1.6: RC 1.6: D 0.2.*—Aug. 1930 to Jan. 1931 with Public Works Eng. Corporation, New York City, as Draftsman, Designer and Jun. Engr., on design of small steel and concrete structures and buildings, and hydrographic and climatological studies, for water-supply projects. *TT 0.4: P 0.4: RC 0.2: D 0.2.*—Jan. to Oct. 1931 with Sanborn and Bogert, Cons. Engrs., New York City, three months as Draftsman and Asst. Designer on sewer and water-supply projects, six months Res. Engr. in charge of construction of sewer and water-supply project for Millbrook, N. Y., including sewers, sewage-disposal plant, water mains, pumping station, etc., also earth fill dam, roads, etc. *TT 0.7: SP 0.1: P 0.6: RC 0.5: D 0.1.*—Oct. 1931 to date Jun. Engr., Pan American Petroleum and Transport Corporation, on surveys for, design and construction of, small structures for bulk oil-storage plants, pump stations, tanks, pipe-lines, truck and tank-car loading racks, buildings, plumbing and heating. *TT 0.4: P 0.4: RC 0.4: D 0.2.*—*TT 8.4: SP 0.4: P 8: RC 3.7: D 1.7.* Refers to C. L. Bogert, R. Kleppe, A. M. McKean, G. M. Neel, C. M. Spofford, T. H. Wiggin, R. R. Wiggins.

The Board of Direction will consider the applications in this list not less than thirty days after the date of its issue.

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PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 58 MAY, 1932 No. 4

ENGINEERING SEISMOLOGY
NOTES ON AMERICAN LECTURES

BY DR. KYOJI SUYEHIRO

LATE DIRECTOR, EARTHQUAKE RESEARCH INSTITUTE
IMPERIAL UNIVERSITY OF TOKYO
TOKYO, JAPAN

At the invitation of the American Society of Civil Engineers, with the financial assistance of John R. Freeman, Hon. M. and Past-President, Am. Soc. C. E., Professor Suyehiro was invited to deliver a series of lectures before educational institutions and scientific societies in America. Through the co-operation of Mr. Freeman, Professor Suyehiro, and the Society, publication of these notes is made possible

Price $1.00 per copy.

FOREWORD

In the fall of 1931 I had the pleasure of visiting the United States at the invitation of the American Society of Civil Engineers, for the purpose of lecturing on Engineering Seismology in some of the prominent American universities. At the kind suggestion of John R. Freeman, Hon. M. and Past-President of the Society, I have arranged my lecture notes in the present form for publication. As every reader is aware, the science of seismology—especially in its application—is far behind other sciences, and is still full of speculations and miscomprehensions. Although I have tried as far as possible to avoid these disputable questions, yet, owing to the great strides that will be made by seismology in the future, I shall not be surprised if much in this booklet will have to be altered and some of it scrapped altogether and rewritten. Indeed, for the well-being of the people of seismic countries, my fervent hope is that this will speedily come to pass.

I take this opportunity to express my sincere thanks to Mr. Freeman for his interest and zeal in the production of this little work and in my lecture tour, for without his efforts nothing could have been accomplished. My gratitude goes to George T. Seabury, the Secretary of the Society, and members of the Administrative Staff, and also to my colleagues and assistants in the Earthquake Research Institute, who have helped me in various ways.

KYOJI SUYEHIRO.

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INTRODUCTION

By John R. Freeman, Hon. M. and Past-President Am. Soc. C. E.

In the following pages the notes are presented from which Professor Kyoji Suyehiro, late Director of the Earthquake Research Institute of the Imperial University of Tokyo, Japan, delivered three lectures in November and December, 1931, at the University of California; Stanford University (California); California Institute of Technology; and Massachusetts Institute of Technology. The lectures were illustrated by lantern slides, most of which have been reproduced.

After graduating in Mechanical Engineering from the Imperial University of Tokyo, and, subsequently, taking special courses in its Physical and Engineering Laboratories pertaining to vibration research, Kyoji Suyehiro became a member of the Engineering Staff of the great ship-building works at Nagasaki. After a period of practical experience, he was called back to the University of Tokyo as Professor of Mechanical Engineering and, later, of Naval Architecture, meanwhile being called into consultation on many practical problems of ship building. His work was recognized to be of such excellence that, in course of time, he was given the degree of Doctor of Engineering.

By reason of his experience in Naval Architecture (which had required intimate study into the action of powerful waves and vibrations on the structure of ships and a familiarity with the means of resisting these stresses and strains by proper structural design, in which the steel framework was of great importance) Professor Suyehiro was particulary well fitted for the direction of researches into stresses caused in structures by earthquake waves, and for determining the requirements of designs for resisting them.

In organizing the work of the Earthquake Research Institute, he sought first to learn more of the character of the earthquake motion and of the forces which produced it. In other words, he sought a definite measure of the character and amplitude of earthquake motion and of its period of vibration. As a means of measuring this force he endeavored to determine its acceleration, which is the commonly adopted means of measuring a physical force. As he states in Lecture I, these measurements of earthquake motion and acceleration have not yet been satisfactorily accomplished; but, meanwhile, improved apparatus and new methods for measuring all effects that appear to have a bearing on the program are being worked out.

Immediately following an important earthquake, the disturbed district is visited by Professor Suyehiro's staff of about a half dozen specialists in various lines of research, who with most painstaking care collect all evidence which seems of importance for further study. In the intervals between such emergencies, these experts are kept busy in their respective fields of physical research, mathematical seismology, geology, and in the devising of new instruments and new means for accurately measuring earthquake motion,

force, and rapidity of vibration. The topographical engineers of the Government co-operate by running lines of precise levels in the district affected, for the purpose of discovering changes of level and earth tilt.

Obviously, in only three lectures of a little more than an hour each, Professor Suyehiro had time only to touch lightly on many of these interesting topics, and the following notes from which he spoke are more condensed than was his oral presentation. A brief synopsis precedes each lecture.

Professor Suyehiro's untimely death on April 9, 1932, creates a distinct loss to the Engineering Profession. His success in interpreting seismology in practical terms has made the entire world his debtor.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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LECTURES

ENGINEERING SEISMOLOGY

NOTES ON AMERICAN LECTURES

By Dr. Kyoji Suyehiro[1]

LECTURE 1

HISTORY OF DEVELOPMENT OF SEISMOLOGY IN JAPAN

Synopsis

Researches undertaken by the Earthquake Research Institute in connection with quakes that occurred in 1923 (the Kwanto), in 1927 (the Tango), and in 1930 (the Northern Idu), are discussed in this lecture. Studies of the Kwanto earthquake have led to the conclusion that, while the upheavals and depressions were, to some extent, the result of gradual changes, most of the changes occurred abruptly at the time of the earthquake. Much investigation is still necessary before scientists can predict earthquake shocks in such a way as to be useful to engineers.

Studies made after the Tango earthquake revealed that the crust of the earth consisted of a number of separate fault blocks. Post-seismic crustal movement was practically ended after four years of re-adjustment. Immediately after the earthquake, Ishimoto tiltgraphs were installed in two places near the epicentral region. A study of the records showed that the occurrence of most of the severe after-shocks were intimately correlated with the change of the direction of the ground tilt; but it has not yet been possible to say whether the tilting was due to meteorological causes or to subterranean changes.

During the Northern Idu earthquake, November 26, 1930, a fault, 30 km. long, appeared in a north-and-south direction through the middle of the Idu Peninsula. Displacements of 100 cm. were measured at the surface, and almost a year afterward daily displacements of 0.001 mm. were observed in the Tanna Railway Tunnel. Many attempts have been made to associate earthquakes with meteorological phenomena and crustal movements.

Note.—These lectures are offered for record and for information and are not open for discussion.

[1] Member, Imperial Academy; Prof., Applied Mechanics, Tokyo Imperial Univ.; Director, Earthquake Research Inst., Tokyo, Japan. Dr. Suyehiro died on April 9, 1932.

Introduction

It is a great honor to me to be invited to lecture at some of the famous universities in this country, and I am delighted to discharge my pleasant task and in that way express the good will of Japan to the United States through science.

First of all, I must emphasize the fact that I am neither a geophysicist nor a geologist (who are the ones best qualified to discuss problems concerning earthquakes), but a seismologist, interested in the study of the engineering phases of seismology.

The fundamental principles of seismology and general surveys of earthquake phenomena are given in Professor J. Milne's "Seismology," Professor R. A. Daly's "Our Mobile Earth," Professor B. Gutenberg's "Handbuch der Geophysik," and other textbooks. Therefore, I shall not enter into general questions, but shall confine my attention to those special problems which have an engineering interest, and which have not yet been discussed in any textbook. Moreover, in a short lecture such as this, it is not possible to describe this very complex subject in any great detail, so I am forced to refer only to some general features of phenomenon.

I wish to call your attention to the fact that my country, Japan, is generally believed to be one of the most seismic countries in the world. The late Professor J. Milne, a famous British seismologist, said in a joke that we have earthquakes for breakfast, dinner, supper, and earthquakes to sleep upon. Needless to say, this is merely a joke and is far from the truth. As a matter of fact, seismicity in my country is somewhat exaggerated, partly because after-shocks of the great 1923 earthquake are still occurring, and partly on account of the fact that whenever a severe earthquake occurs, it is known immediately throughout the world, because Japan is so densely populated that more or less damage is wrought by every severe earthquake, wherever it may take place.

You are in a quite different situation. For instance, you had a terrible earthquake in Northern Nevada in 1915, which left a remarkable fault having a displacement of about 20 ft., vertically, on the west side of the Sonoma Range, but so far as I am informed, only one ranch house close to the fault was seriously injured, and there was no damage to other structures because these were in general non-existent. In Japan we cannot dream of such a situation. I do not deny, however, that earthquakes are frequent in my country. Indeed, for that reason, we are doing our utmost to understand and conquer them; but I am sorry to say that our efforts thus far have not been fruitful.

The foundations of the present Japanese seismology were laid by a corps of American and English scientists who came to Japan about five decades ago. In 1880, these foreign scientists established the Seismological Society of Japan and studied seismological problems very eagerly. Among these men we may specially mention the names of J. Milne, C. G. Knott, J. A. Ewing, and T. C. Mendenhall. Professor Ewing succeeded in obtaining

FIG. 1.—VIEW OF TOKYO, JAPAN, TAKEN SOON AFTER THE KWANTO EARTHQUAKE, SEPTEMBER 1, 1923.

the registration of horizontal components of earthquake motions by a newly invented horizontal pendulum. Professor Milne devoted himself with the utmost eagerness to the study of the nature of earthquakes, and fostered the development of this branch of science. We are very grateful for the work done by these eminent scientists.

When these men returned home, the work was taken up by the late Professor K. Sekiya and the late Professor F. Omori, most of their study being directed toward near-by earthquakes, which is characteristic of Japanese seismology.

The earthquake of October 28, 1891, in the Provinces of Mino and Owari, situated in the central part of the main island of Japan, was the first destructive earthquake to occur after we took up the Western science, and it caused the Japanese people to realize vividly the necessity of scientific studies.

The next year a Government institution, called the Earthquake Investigation Committee, was established for the study of both the scientific and engineering problems of earthquakes. Leading physicists, geologists, and engineers were appointed members and did splendid work, reports of which were published in the *Proceedings* of the Earthquake Investigation Committee.

Above all, the investigations of earthquake-resisting structures were most important. Buildings constructed to comply with the Committee's recommendations withstood the 1923 earthquake without damage.

After the 1891 earthquake, Japan enjoyed freedom from severe disturbances for more than thirty years, until, in 1923, the recent great earthquake took place. Forgetfulness is a trait of human nature. During the years that followed the earthquake of 1891, the members of this first scientific committee one by one returned to their customary vocations, leaving as the only continuous worker, Professor Omori, who, unfortunately, did not live to investigate the great earthquake of 1923.

A renewed interest in the study of earthquakes followed the great shock of 1923, and, in 1925, the Earthquake Research Institute was established at the Tokyo Imperial University, the active part of the work of the Earthquake Research Committee being transferred to this Institute.

Since its establishment, there have been two severe earthquakes, the Tango and the Idu. Our investigations of these two severe disturbances will be mentioned briefly with that of 1923. The great Kwanto earthquake, which took place on September 1, 1923, destroyed the greater part of the City of Tokyo as a direct result of fires that followed. The Tango earthquake occurred on March 7, 1927, in the central part of the main island of Japan, facing the Japan Sea. The Idu earthquake took place on November 26, 1930, and devastated the northern part of the Province of Idu, a peninsula projecting southward into the Pacific. The last two were quite local in character, but some of the damage was no less severe than in the Kwanto earthquake.

FIG. 2.—VERTICAL DISPLACEMENT OF THE GROUND IN THE AREA DISTURBED BY THE KWANTO EARTHQUAKE

Fig. 2.—Vertical Displacement of the Ground in the Area Disturbed by the Kwanto Earthquake

(I) The Kwanto Earthquake

This destructive earthquake is too well known to require any special mention, but I will just show by an airplane view taken soon afterward (Fig. 1) how the capital of Japan, although it was not situated in the zone of greatest shaking, was practically destroyed by the fires that followed. I will mention briefly some of the researches undertaken on this earthquake.

1.—Vertical Displacement of the Ground.—The result of precise levels taken by the Military Land Survey Department is shown in the map (Fig. 2). The work subsequent to the earthquake was done during the period between September, 1923, and March, 1927. The vertical displacement of the land was determined by comparing these results with data obtained from similar work done between 1888 and 1903.

The earth's crust forming the Japanese Islands, is remarkably unstable, even at ordinary times, so there is no doubt that the upheaval and depression that have been found, consist of both constant, gradual changes, and abrupt changes. Most of the changes occurred abruptly at the time of the earthquake, as has been clearly proved not only by the tide gauge record, or mareogram, taken at Yokosuka (Fig. 3), a port situated in the most violently shaken district, but also by newly raised beach lines in the devastated area.

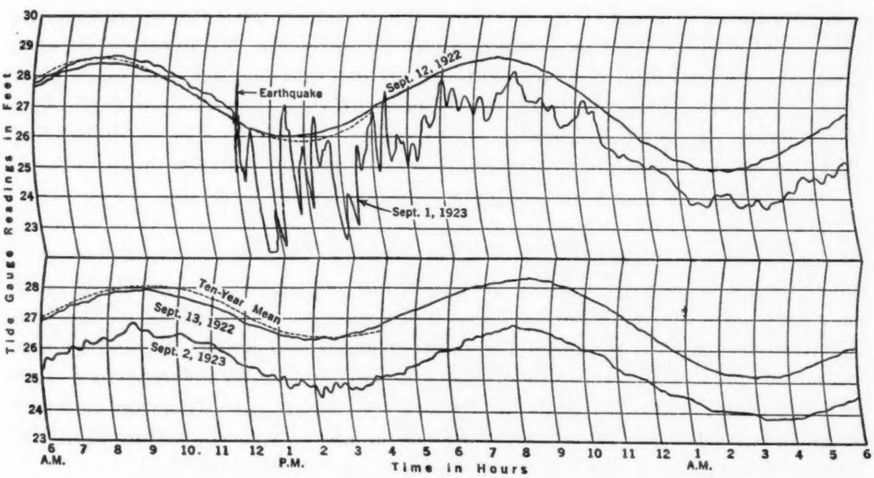

Fig. 3.—Tide-Gauge Readings at Yokosuka, Showing that Changes in Elevations Occur Abruptly.

Incidently, from a mareogram taken at Aburatubo, a village not far from Yokosuka, it was found that nearly a month previous to the earthquake the ground apparently began to upheave slowly relative to the sea level. Just before the occurrence of the earthquake, the total rise had amounted to about 2 cm. Some believed this uplift was the harbinger of the earthquake, and that means for predicting earthquakes might be obtained from such crust movements.

However, by a careful study of this phenomenon, Professor T. Terada, of our Institute, concludes that such an occurrence of pre-seismic upheaval relative to the sea level might have been due to meteorological causes. This would mean that the changes of the sea level were due to changes in barometric pressure. At the time, typhoon centers passed over the Pacific, one after another, south of the Japanese Islands. Taking this fact into consideration, Professor Terada showed that the earth's crust was not disturbed just before the earthquake. Thus, we might be hasty if we conclude that evidence of such a crustal movement can be depended upon to foretell an earthquake. There is still much to be investigated before we can hope to predict earthquake shocks. This question will be discussed later.

The second map (Fig. 4) shows the vertical displacements of benchmarks along the principal leveling traverses in the western half of the main island of Japan. Naturally, the land disturbance is most marked in the following three regions: The Kwanto District, the Tango Peninsula, and the Idu Peninsula, where great earthquakes have occurred recently. It will be seen, however, that there are a number of other places which are thus far intact, which have experienced decided crustal movements. We should watch closely the progress of these gradual changes.

Reverting to the subject of crust movements that followed the earthquake, analyses made in the Earthquake Research Institute, gave several interesting results. I will mention only one of these made by Mr. N. Miyabe. He analyzed the vertical land displacements by his original method, and made clear that the vertical movements of the Boso Peninsula were composed of block movements, the general features of which are shown in Fig. 5, indicating that the crust in this district probably was of mosaic construction.

His method is based on a simple trigonometrical relation,

$$\tan \phi = \tan \phi_m \cos (\theta - \theta_m) \qquad \text{(1)}$$

which holds when any number of points on one and the same block move *en masse*. In the equation, ϕ denotes the change in the slope of the line that connects a pair of points (or relative change in height divided by distance); θ denotes the azimuth of this connecting line as referred to a definite direction, while ϕ_m and θ_m, respectivly, denote the direction and the magnitude of the change in slope of the block. Using Equation (1), Mr. Miyabe distinguished the different blocks by a graphical method.

He plotted the values of $\tan \phi$ of various points and their relations to θ, and segregated the groups of points that lie on one and the same block from the others, by judging whether or not they fell on a fairly well-defined sinusoidal curve.[2]

In order to determine how the crust movements progressed after the great convulsion, precise levels were repeatedly run along a line from Tokyo and Aburatubo, by the Military Land Survey Department, at the joint request of the Earthquake Advisory Committee and Professor A. Imamura.

[2] For details, refer to paper by N. Miyabe in *Bulletin*, Earthquake Research Inst., Tokyo Imperial Univ., Vol. 9, No. 3.

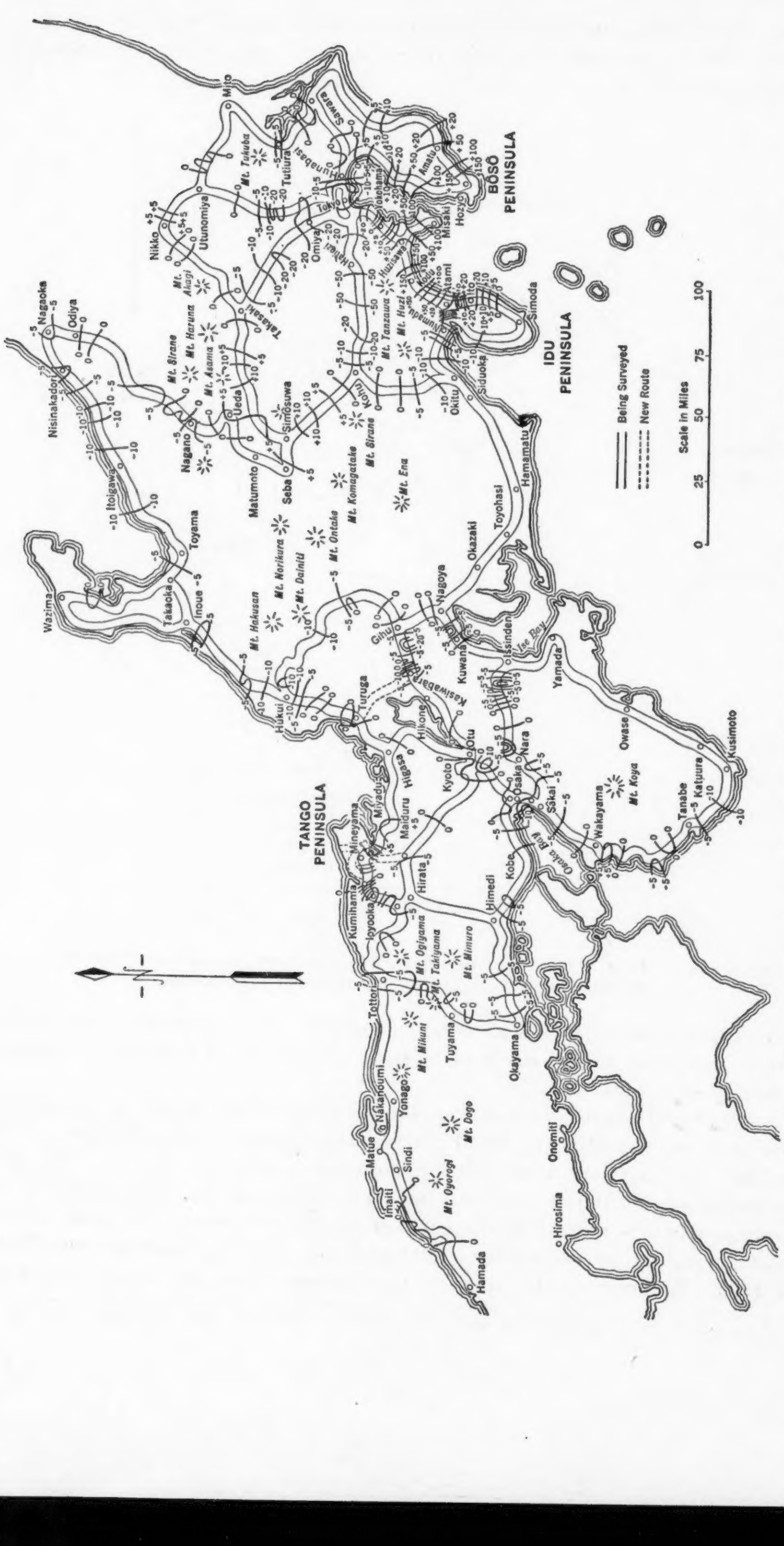

FIG. 4.—VERTICAL DISPLACEMENT OF BENCH-MARKS IN THE WESTERN HALF OF THE ISLAND OF HONSHU.

It was found that immediately after the earthquake the crust had moved in such a way as to resume its original level, but it was afterward subject to further uplift.[3]

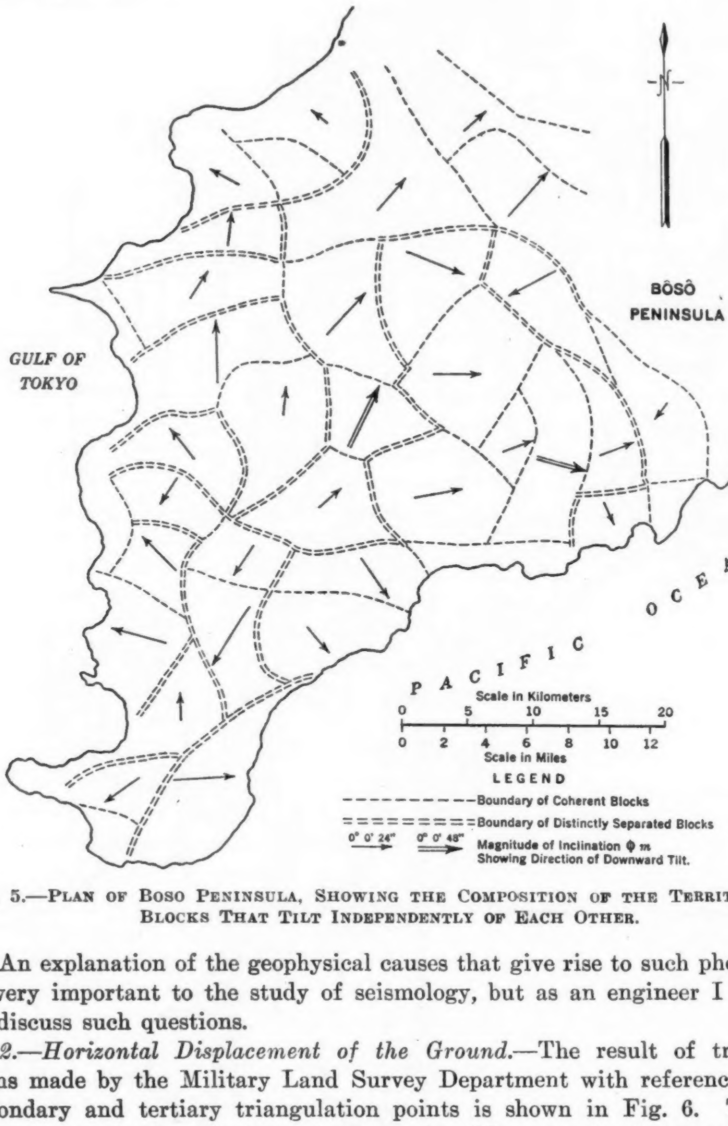

FIG. 5.—PLAN OF BOSO PENINSULA, SHOWING THE COMPOSITION OF THE TERRITORY INTO BLOCKS THAT TILT INDEPENDENTLY OF EACH OTHER.

An explanation of the geophysical causes that give rise to such phenomena is very important to the study of seismology, but as an engineer I hesitate to discuss such questions.

2.—Horizontal Displacement of the Ground.—The result of triangulations made by the Military Land Survey Department with reference to 778 secondary and tertiary triangulation points is shown in Fig. 6. The displacements of triangulation points are the differences in their positions as determined before and after the earthquake, the former having been determined during the period between 1890 and 1891 and the latter between 1924 and 1926. In drawing this map, it was assumed that the heavy boundary

[3] See paper by A. Imamura, *Proceedings*, Imperial Academy, Vol. 6. (1930), No. 10.

lines connecting the principal primary points had not suffered any disturbance. This assumption was made for convenience in simplifying the work of revising the map, so that the displacements shown are to be understood as being only relative. Nevertheless, the map does not fail to impress one with the catastrophic nature of the earthquake.

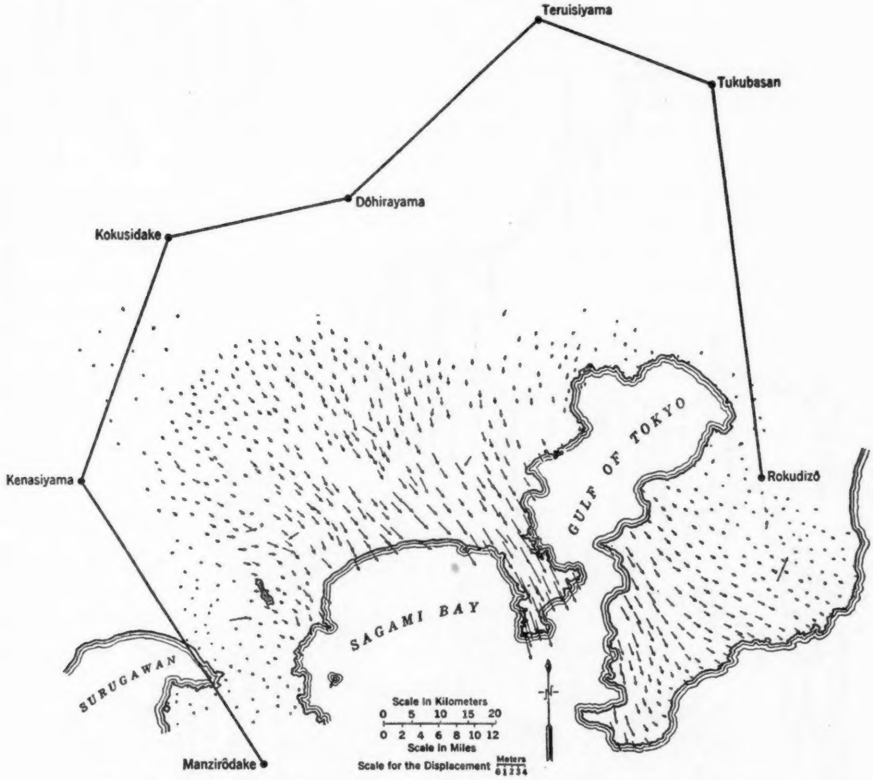

FIG. 6.—RESULT OF SURVEY TO DETERMINE HORIZONTAL DISPLACEMENTS.

3.—*Disturbance in the Sea Bed.*—The disturbance that occurred on the sea bottom of Sagami Bay was far more severe. Fig. 7 shows the differences in the depths between the soundings made by the Naval Hydrographic Department immediately after the earthquake, during the period between September, 1923, and February, 1924, compared with those made in 1912 and thereafter.

Since the apparent changes in depth amounted to as much as 200 m. in several places, one may well doubt the accuracy of the soundings. However, the soundings not only were made by highly trained men, but the results have been carefully checked by Professor Terada for the purpose of determining whether the reported changes were independent of the declivity of the slope of the sea bed or of the determined depths of the bed. He found that the results of the measurements were not unreliable. The fact that when

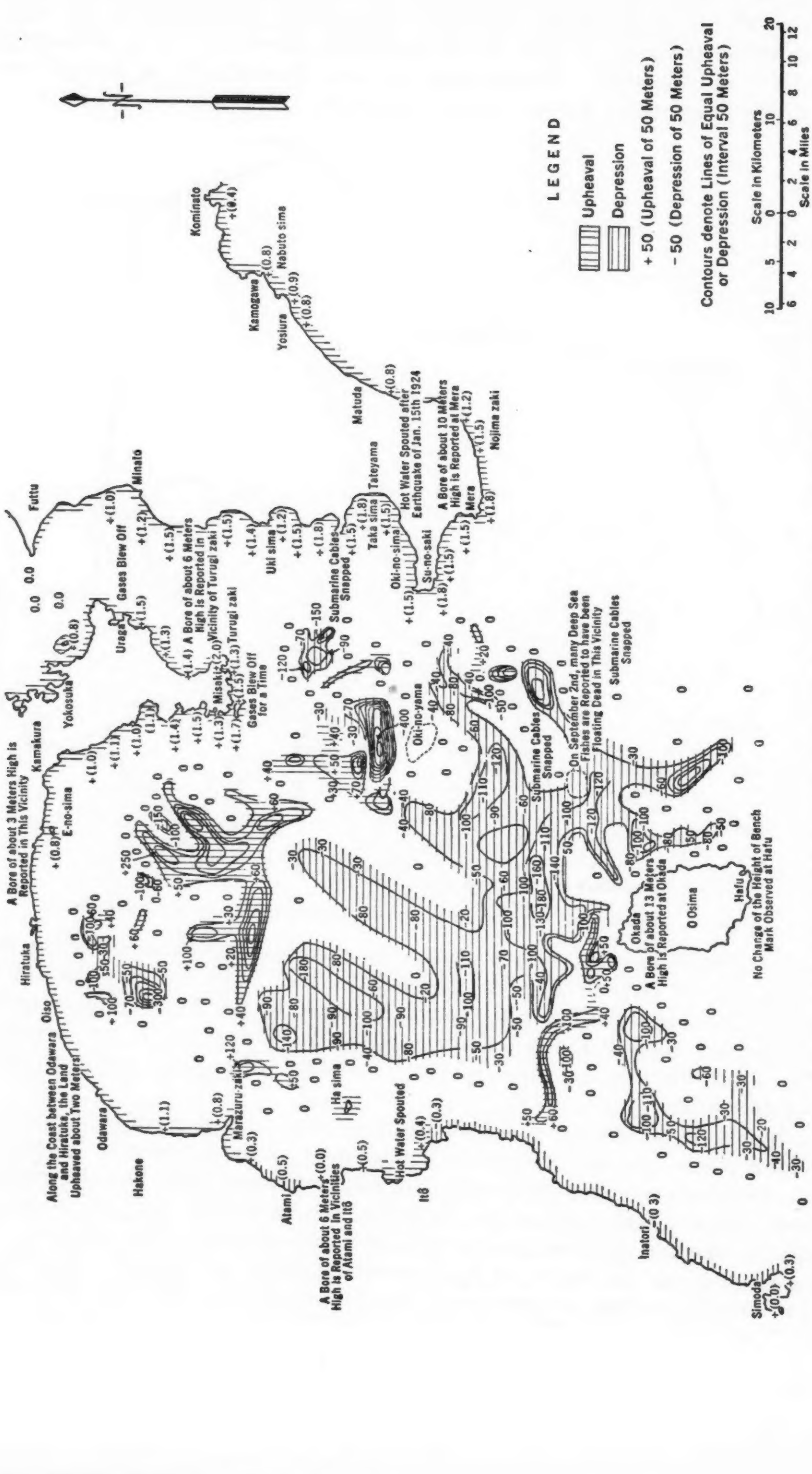

FIG. 7.—CHANGES IN DEPTH OF SAGAMI BAY AND VICINITY BETWEEN 1912 AND 1923 (IN METERS).

—on April 5, 1930,—a few points were sounded a second time, they gave practically the same readings, seems to prove further the reliability of the soundings. However, in view of the additional fact that in the seaquake of August 15, 1886, near Zante, Greece, a sudden increase of from 4 500 to 5 800 ft. in the depth of the sea bed was reported, one might be convinced that the sea bed behaves quite differently from dry land in an earthquake.

Although without any direct connection with the problem under consideration, it is worth mentioning that while taking soundings in the Pacific Ocean, south of Japan, the Naval Surveying Ship, the *Mansui,* found a deep yawning chasm along the dotted islands of Idu, having a depth far in excess of the well-known Tuscarora Deep.

4.—Changes in the Length of the Ground.—Crustal deformation was again well illustrated by the changes in the lengths of periodic base lines established on the grounds of the Astronomical Observatory of the Tokyo Imperial University at Mitaka, a village just outside of Tokyo. The results of measurements that have been made annually by the Geodetic Commission have been plotted and are shown in Fig. 8. The changes in the area of the

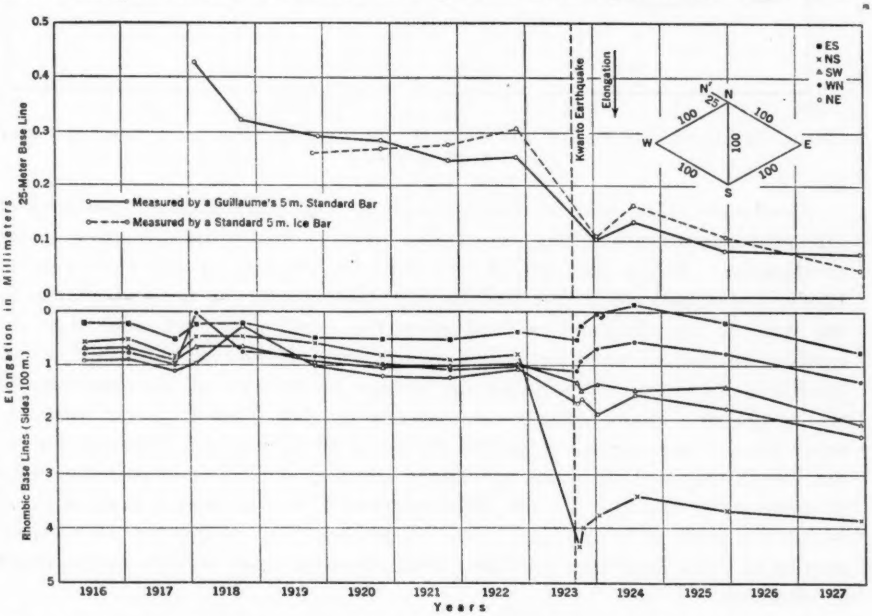

FIG. 8.—MEASUREMENTS OF BASE LINE ON THE GROUNDS OF THE ASTRONOMICAL OBSERVATORY OF TOKYO IMPERIAL UNIVERSITY, SHOWING CHANGES OF LENGTH.

geodetic rhombus computed from these data by Mr. C. Tsuboi are shown in Fig. 9. The fact that the peculiar compressional change of area which was going on just before the earthquake was succeeded by an enormous expansion just after the shock, deserves attention.

I may add that we are now preparing to provide for taking continuous observations of the changes in measured distances at one of our observation

stations. For this purpose, quartz tubes, 20 m. long, installed in covered trenches, will be used as standard scales of reference. It is expected that observations will begin some time during 1932.

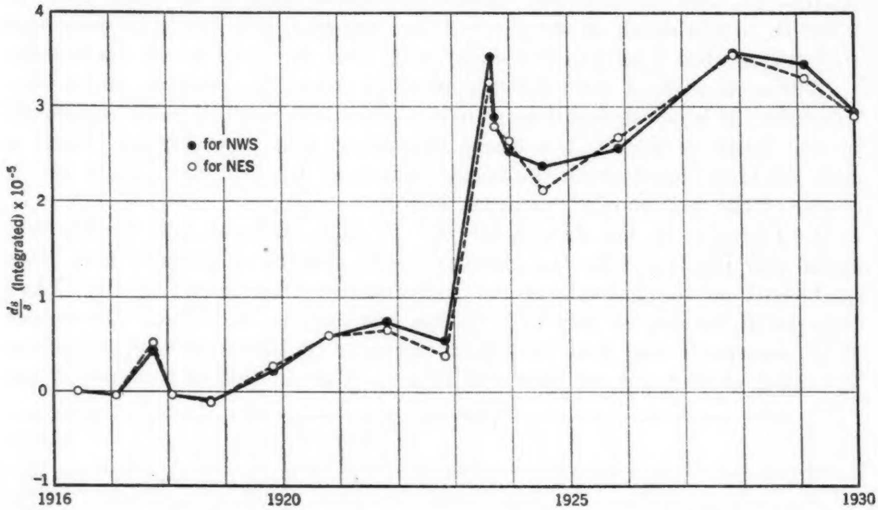

FIG. 9.—RELATIVE CHANGES IN THE AREA OF THE GEODETIC RHOMBUS, COMPUTED FROM DATA IN FIG. 8.

5.—Acceleration Due to Gravity.—There were various phenomena which attracted our attention, but were found to have no intimate relation to the earthquake. As an example, I will cite the change in the time rate of Riefler's standard clock installed in the Observatory. A decided change in the rate of the clock before and after the earthquake indicated that the earthquake had changed the value of the gravitational acceleration in this locality. To determine whether the change in the rate of the clock was in reality due to a change in the value of gravity, Mr. Tsuboi, of our Institute, made gravity measurements, in the spring of 1931, with the Nagaoka Tungsten Pendulum in Tokyo and at distant stations, well beyond possible range of earthquake effects. To our disappointment it was found that, at least within the order of 10 milligal[4] (0.01 cm. per sec.^{2}), no change in the value of gravity at three places, as compared with pre-earthquake measurements, could be observed.

Such investigations should have been made immediately after the earthquake. I regret that on account of the limited funds available, we are continually missing good opportunities for studies of this nature. We are really disappointed that our contributions to the development of this branch of science are relatively incomplete although, whether fortunately or unfortunately, we have always been, and no doubt always will continue to be, well supplied with abundant material for investigation.

[4] 1 milligal=1/1000 gal.=1/1000 of the centimeter-gramme-second unit of acceleration; 1 gal.=g/980=1/980 of the foot-pound-second unit of acceleration.

(II) The Tango Earthquake

After the Tango earthquake—in addition to the usual routine work, consisting of seismometric observations of after-shocks, geological and geographical surveys, soundings of sea beds, and engineering inspections, the following work was undertaken:

(1) Precise levels run over the disturbed area were repeated five times, the last series having been completed in the fall of 1930.
(2) Triangulation traverses in the same area were re-run three times.
(3) Immediately after the earthquake, Ishimoto tiltgraphs were installed in two places near the epicentral region, and tilting of the ground was observed continuously.

1.—Precise Leveling.—The routes of the survey are shown in Fig. 10, in which the approximate locations of the two principal fault lines, or rather the fault echelons, have been located. The five sets of profiles, showing the

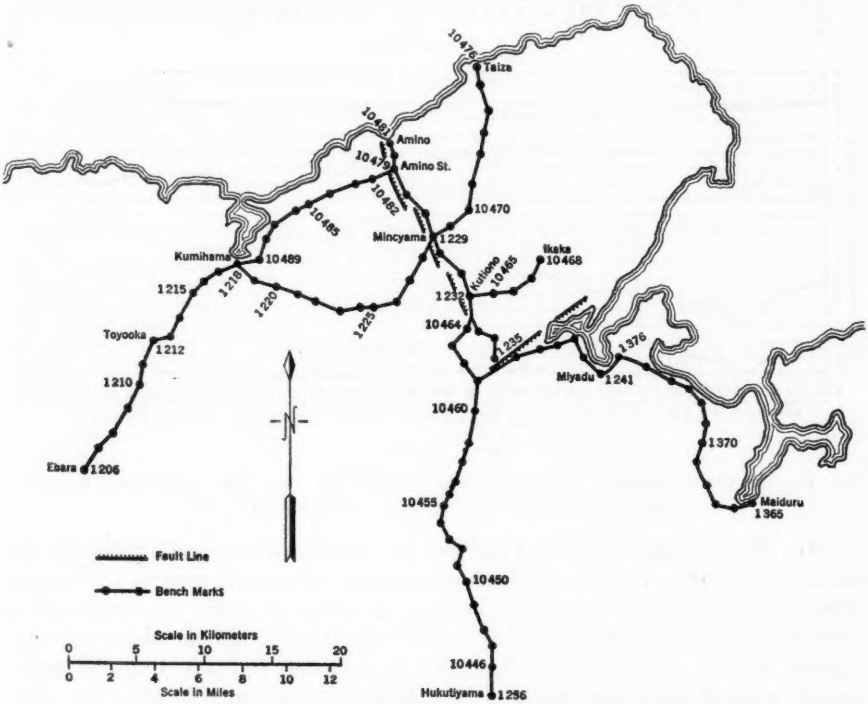

Fig. 10.—Leveling Traverses for Use in Observing Effects of the Tango Earthquake.

upheaval and the depression that occurred during the interval between successive surveys along one of the routes, are embodied in Fig 11. It is interesting to note that during the early stages of crustal re-adjustment, both sides of the fault lines moved independently toward each other, but later behaved as one block—especially the north and south lines—suggesting that the fault fissures were consolidating; and also that in the course of settling,

the disturbed areas moved upward at one time and downward at another, the movements gradually fading out. Since in the profiles the curved routes are expanded into straight lines, it is somewhat difficult to visualize the real nature of the crustal movement.

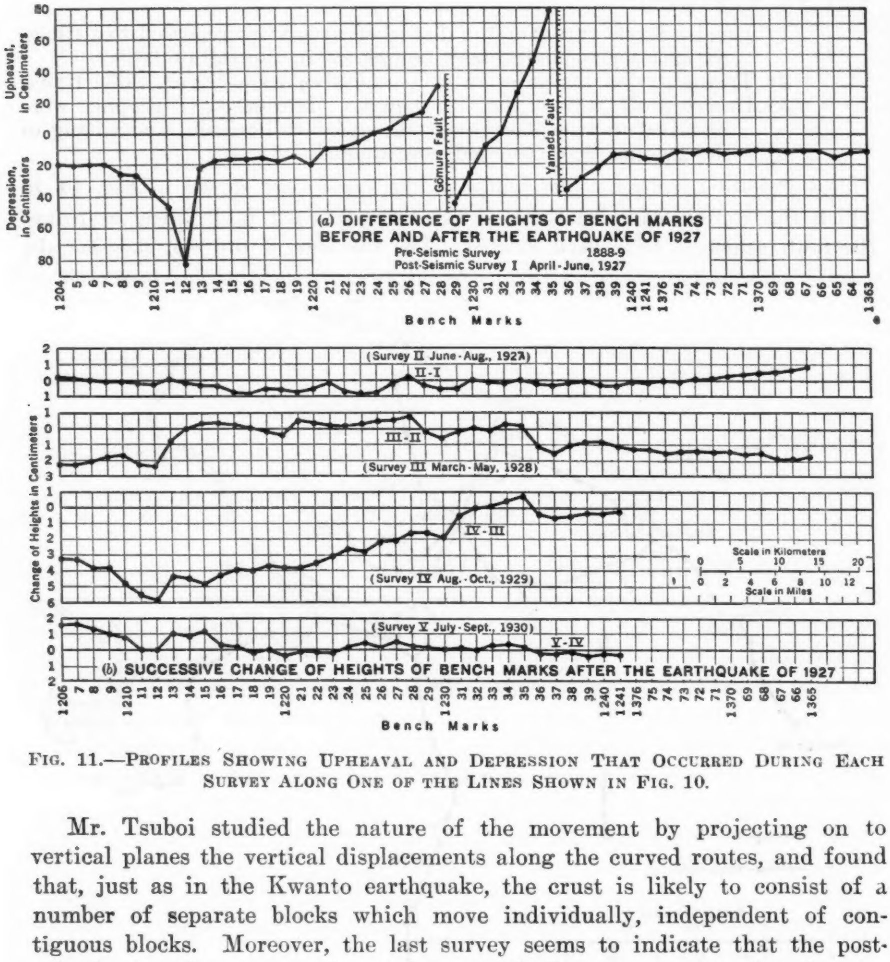

FIG. 11.—PROFILES SHOWING UPHEAVAL AND DEPRESSION THAT OCCURRED DURING EACH SURVEY ALONG ONE OF THE LINES SHOWN IN FIG. 10.

Mr. Tsuboi studied the nature of the movement by projecting on to vertical planes the vertical displacements along the curved routes, and found that, just as in the Kwanto earthquake, the crust is likely to consist of a number of separate blocks which move individually, independent of contiguous blocks. Moreover, the last survey seems to indicate that the post-seismic crustal movement has at last practically come to rest after the continual re-adjustments that went on for four years.

2.—Triangulation.—After the earthquake, triangulation was also repeated three times, between May and June, between August and September, and between October and November, 1927. The pre-seismic surveys were made between 1884 and 1889. The displacements of principal triangulation points during each successive survey—taking the position of a point farther south of the affected area, as well as the direction of a line connecting this point to another distant point, as fixed—are shown in Fig. 12.

The diagram may be valuable for studying the nature of earthquakes. However, I refrain from discussing the subject, and will only say that several important investigations based upon these surveys have been made by members of our staff, such as that by Professor Terada on the distribution of the dilatation and the rotation of the ground over the disturbed area, the results of which have been published in the *Bulletins* of the Earthquake Research Institute of the Tokyo Imperial University.

3.—Tilting of the Ground.—The Ishimoto tiltgraphs were used for the first time after the Tango earthquake, to observe post-seismic crustal movements. This instrument is essentially a horizontal pendulum with Zöllner's

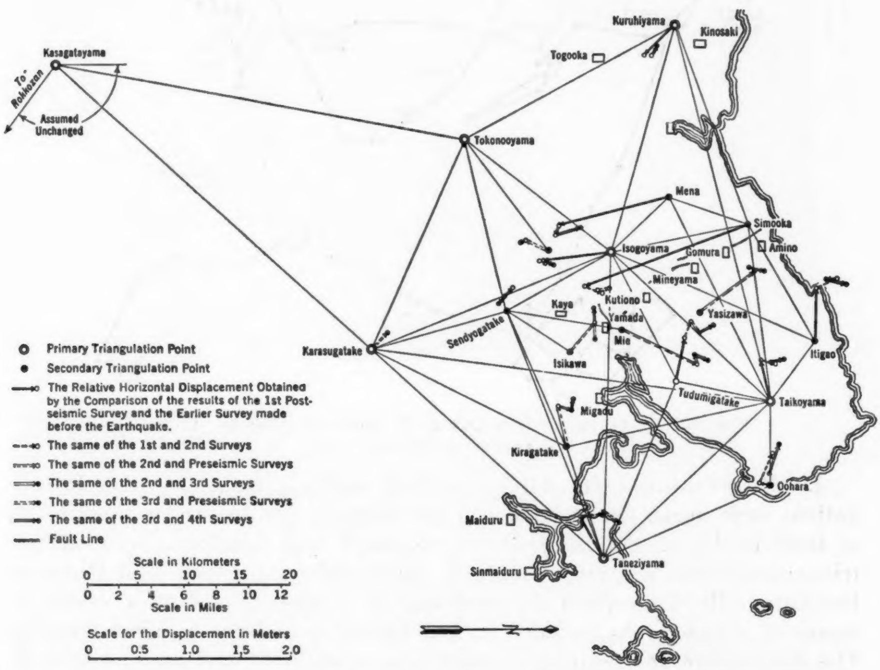

FIG. 12.—HORIZONTAL DISPLACEMENT OF TRIANGULATION POINTS OBSERVED AFTER THE EARTHQUAKE IN THE TANGO DISTRICTS, MARCH 7, 1927.

suspension and is made of fused silica throughout, in order to reduce to a minimum the effects of temperature variation. It indicates ground tilts as small as 0°-0′-0.1″. The instruments were installed in two places in the disturbed region. The important after-shocks seem to be closely related to the tilting of the ground. This is shown in the vector diagram (Fig. 13) of the ground tilts, as recorded by the tiltgraph. An Ishimoto tiltgraph is shown in Fig. 14.

It is seen that the occurrence of most of the strong after-shocks (marked by a ringed dot) was intimately correlated with the change of the direction of the ground tilt. We are not sure, however, whether these tiltings were

due to meteorological causes or to subterranean changes. Whatever the cause, we are not yet in a position to say that such tilting movements do unmistakably foretell an earthquake.

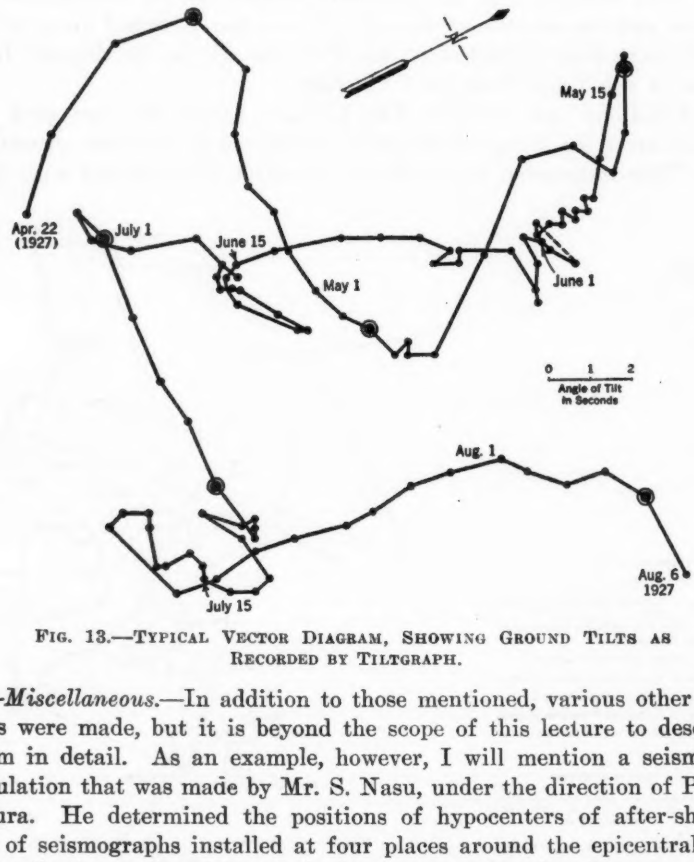

FIG. 13.—TYPICAL VECTOR DIAGRAM, SHOWING GROUND TILTS AS RECORDED BY TILTGRAPH.

4.—Miscellaneous.—In addition to those mentioned, various other investigations were made, but it is beyond the scope of this lecture to describe all of them in detail. As an example, however, I will mention a seismometric triangulation that was made by Mr. S. Nasu, under the direction of Professor Imamura. He determined the positions of hypocenters of after-shocks by means of seismographs installed at four places around the epicentral region. The distribution and position of the hypocenters thus found may prove to be useful data for seismologists.

(III) THE NORTHERN IDU EARTHQUAKE

Since investigations of this destructive earthquake are still in progress (September, 1931), the report will be somewhat incomplete.

1.—General Descriptions.—This earthquake, although very local in character, was destructive, causing a loss of 261 lives and the destruction of more than 2 000 houses. According to Professor Imamura, the origin of this earthquake was in Longitude 139°.0 E. Latitude 35°.2 N. A great fault 30 km. long appeared along a well-known tectonic line in a general north and south direction, and running through the middle of the Idu Peninsula. The land east of the fault was displaced northward relative to that lying to the west

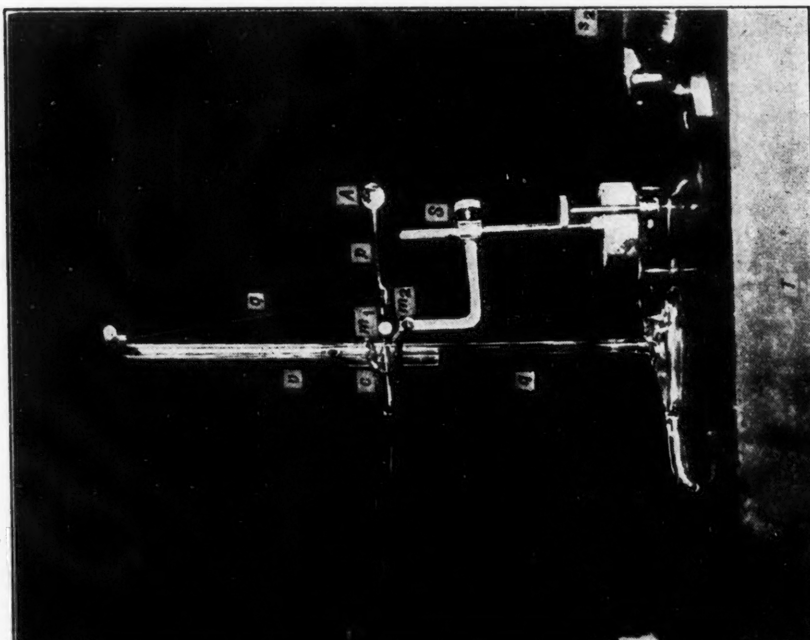

FIG. 14.—AN ISHIMOTO TILTGRAPH.

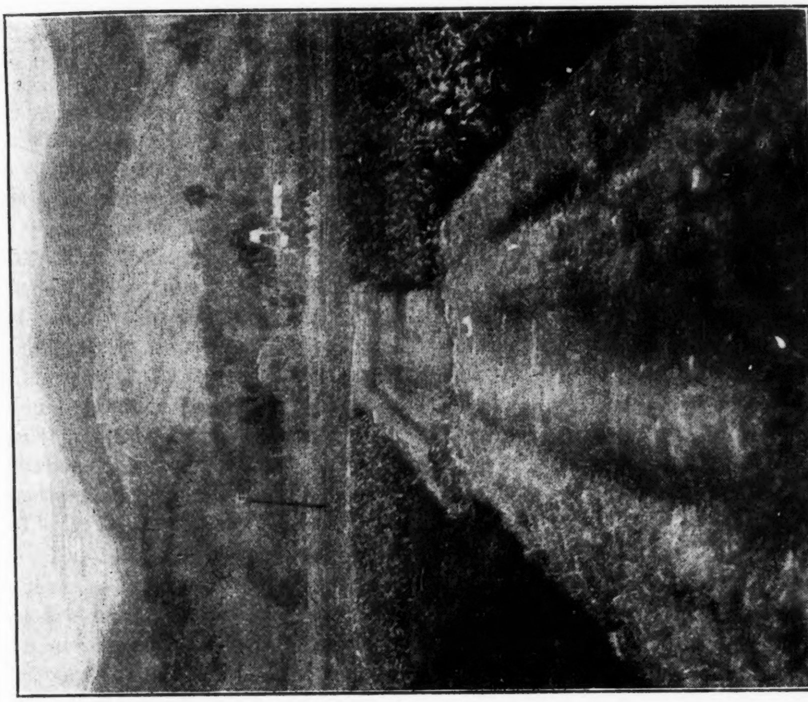

FIG. 15.—SEVERE EARTH DISPLACEMENT THAT OCCURRED IN MIDDLE OF THE IDU PENINSULA.

of it, the horizontal displacement being about 100 cm., as measured on the ground surface in the Tanna Basin, where the fault was well developed. (Fig. 15). This displacement phenomenon was especially interesting in that it was also observable inside the Tanna Railroad Tunnel, as will be described in detail later.

2.—Fore-Shocks and After-Shocks.—A remarkable feature of this earthquake was the great number of fore-shocks that began as early as November 7, preceding the main earthquake, and culminating in a shock on November 26, which attained destructive intensity. According to the Central Meteorological Observatory, 789 shocks were recorded at its observing station at Misima on the day previous to the occurrence of the main earthquake. A rapid succession of shocks, or earthquake "swarms," also began to occur on February 13, 1930, from the sea bed off Ito, a spa on the western coast of the Idu Peninsula, situated about 10 km. from the epicenter of this destructive earthquake. These earthquake "swarms" may also be regarded as fore-shocks. Perhaps they hold the record in frequency of occurrence, 3 715 shocks with motions exceeding 1 micron having occurred between February 14 and April 11.

The three-dimensional distribution of the hypocenters of the Ito earthquakes as determined by Mr. Nasu and others, with five seismographs installed at five places encircling the epicentral region, is shown in Fig. 16. It will be seen that the foci are clustered within a small, horn-shaped volume. Consequently, some are of the opinion that this seismic activity was due to the action of some dormant submarine volcano.[5]

3.—Ground Tilting Caused by Ito Earthquakes.—Ishimoto tiltgraphs were installed at two places (Ito and Kawana), near the epicentral region, in order to find the correlation, if any, between the titling motion of the ground and the earthquakes. Fig. 17 is a vector diagram showing the direction and magnitude of the ground tilts observed at Ito. The conspicuous shocks with their dates of occurrence are marked, the former with letters of the alphabet and the latter with numerals.

It will be seen that while one group of shocks was associated with a ground tilt toward the west, another group was associated with a tilt toward the south. During April, there was a period of quiet when tilting movements were hardly in evidence—possibly "marking time", thus appearing to confirm the seeming relationship of the crust tilts to earthquakes. It should be noted, however, that although the earthquake of March 22 (marked *K* in the diagram), was very destructive, nothing like a warning of its coming was ever noticed.

Partly through unavoidable circumstances, and partly through lack of thought on my part, observations were discontinued in October. Unfortunately, the destructive earthquake of November 26 occurred before we had resumed observations as contemplated, thus causing us to miss excellent opportunities of observing crustal movements just before and after a destruc-

[5] "Recent Seismic Activities in the Idu Peninsula", by S. Nasu, F. Kishinouye, and T. Kodaira, *Bulletin*, Earthquake Research Inst., Vol. 9 (1931), No. 1.

tive earthquake, although, as we have subsequently learned, we could not have accomplished much because of the fact that the crustal movements connected with the main earthquake in this particular district were not at all conspicuous.

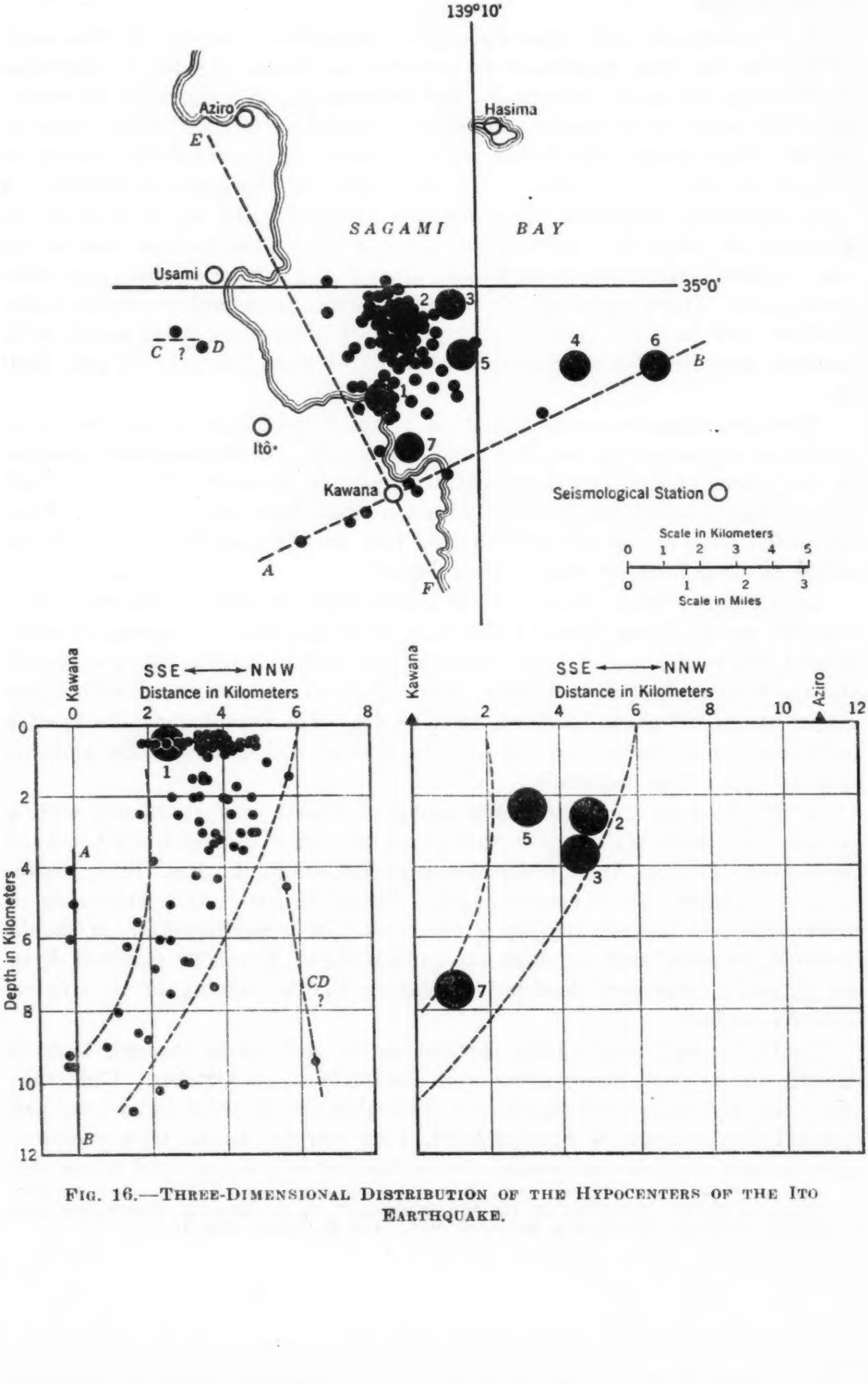

FIG. 16.—THREE-DIMENSIONAL DISTRIBUTION OF THE HYPOCENTERS OF THE ITO EARTHQUAKE.

4.—Precise Leveling and Triangulation.—On the other hand, we had seen the necessity of running a line of precise levels along a route passing close to the epicentral region of the Ito earthquakes, as shown in Fig. 18. The work was entrusted to the Military Land Survey Department and the survey was repeated along the same route, while the shocks were still going on.

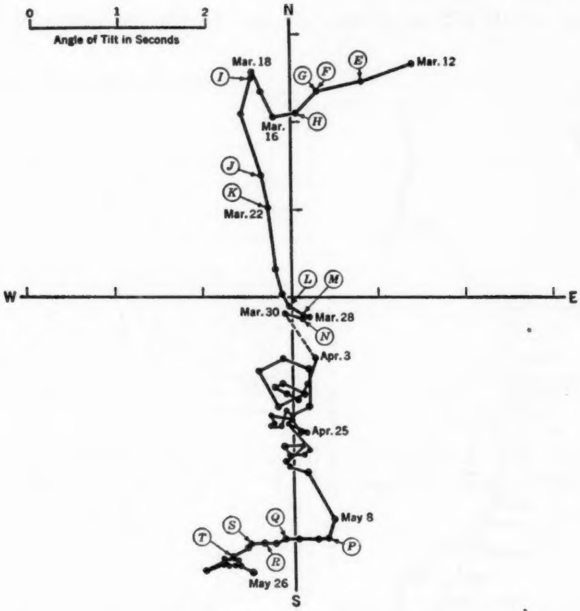

FIG. 17.—VECTOR DIAGRAM SHOWING THE DIRECTION AND MAGNITUDE OF GROUND TILT, OBSERVED AT ITO.

Thus—together with the surveys made by the Survey Department both before and after the Kwanto earthquake, and after the severe Idu earthquake on November 26—precise levels were repeated five times along this route. The vertical displacements observed after each survey as referred to previous findings are shown in Fig. 18.

The following points are worth noting:

(*a*) The destructive earthquake occurred while our second survey (marked II) was being carried out. Indeed, the survey between Bench-Marks Nos. 9 339-9 341, was made in one direction on November 25, the day previous to the occurrence of the earthquake, and in the opposite direction on the next day, just after the earthquake; no appreciable change, however, was observed.

(*b*) During the short interval of seven months between our first and second surveys, the coastal bench-marks facing the epicenter from which the seismic group originated upheaved enormously.

In addition to those just mentioned, the Land Survey Department, immediately after the destructive earthquake, ran precise levels around the Idu Peninsula, the northern route of which crosses a fault line. The vertical

displacements, referred to those of the previous survey, are shown in Fig. 19. Analyses and interpretations of these results were made by Mr. Tsuboi and other members of the Staff of the Institute, and were published in the *Bulletins*.

Triangulation surveys over twelve primary points and seventeen secondary points in the Idu Peninsula and environs were made in the summer of 1931, the results of which will soon be published in the *Bulletins* of the Institute.

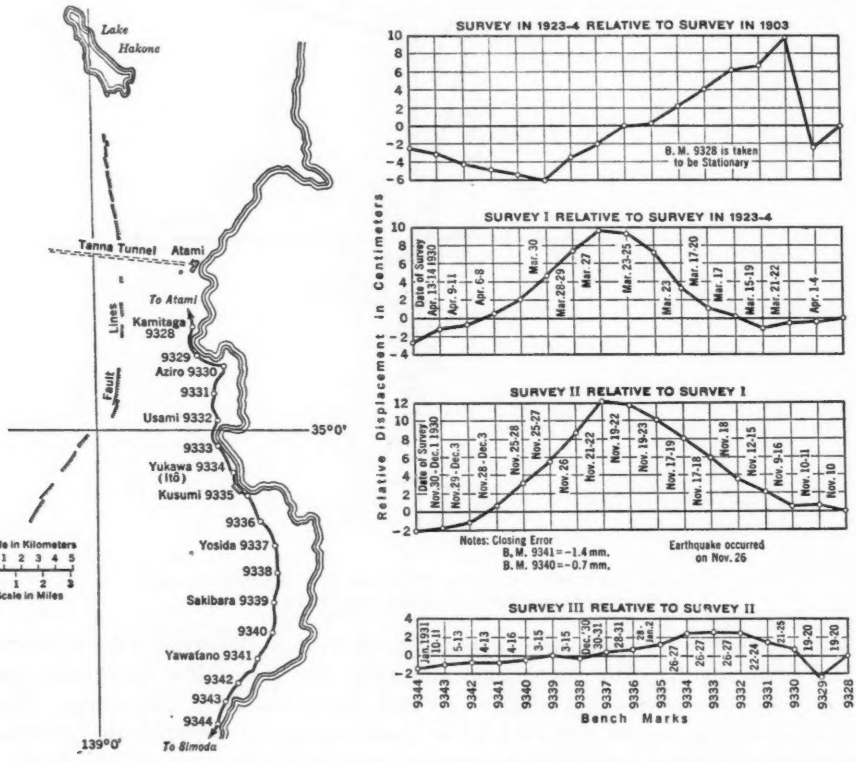

FIG. 18.—PRECISE LEVELS ALONG A ROUTE PASSING CLOSE TO THE EPICENTRAL REGION OF THE ITO EARTHQUAKE.

5.—The Tanna Tunnel.—In the location shown roughly in Fig. 18, a railroad tunnel having a total length of nearly 26 000 ft. is being bored in a general east and west direction. At the time of the earthquake, about 3 000 ft. remained unbored. (More detailed information is given in Fig. 20.) It will be seen that all fault planes are in a meridional direction, and that they crossed the tunnel perpendicularly. Of the four fault planes in the tunnel, the displacements of three were not serious, since they resulted in only slight cracks in the tunnel wall or small steps in the floor. One of them, which is probably a continuation of the main fault that appeared outside on the Tanna Basin, was very remarkable.

At the time of the earthquake, tunneling work from the west portal had proceeded to a point 11 920 ft. distant, where a muddy water-logged layer was encountered. It was along this muddy layer that the most remarkable dislocation occurred. At one end of the drain tunnels a sort of "slickenside"

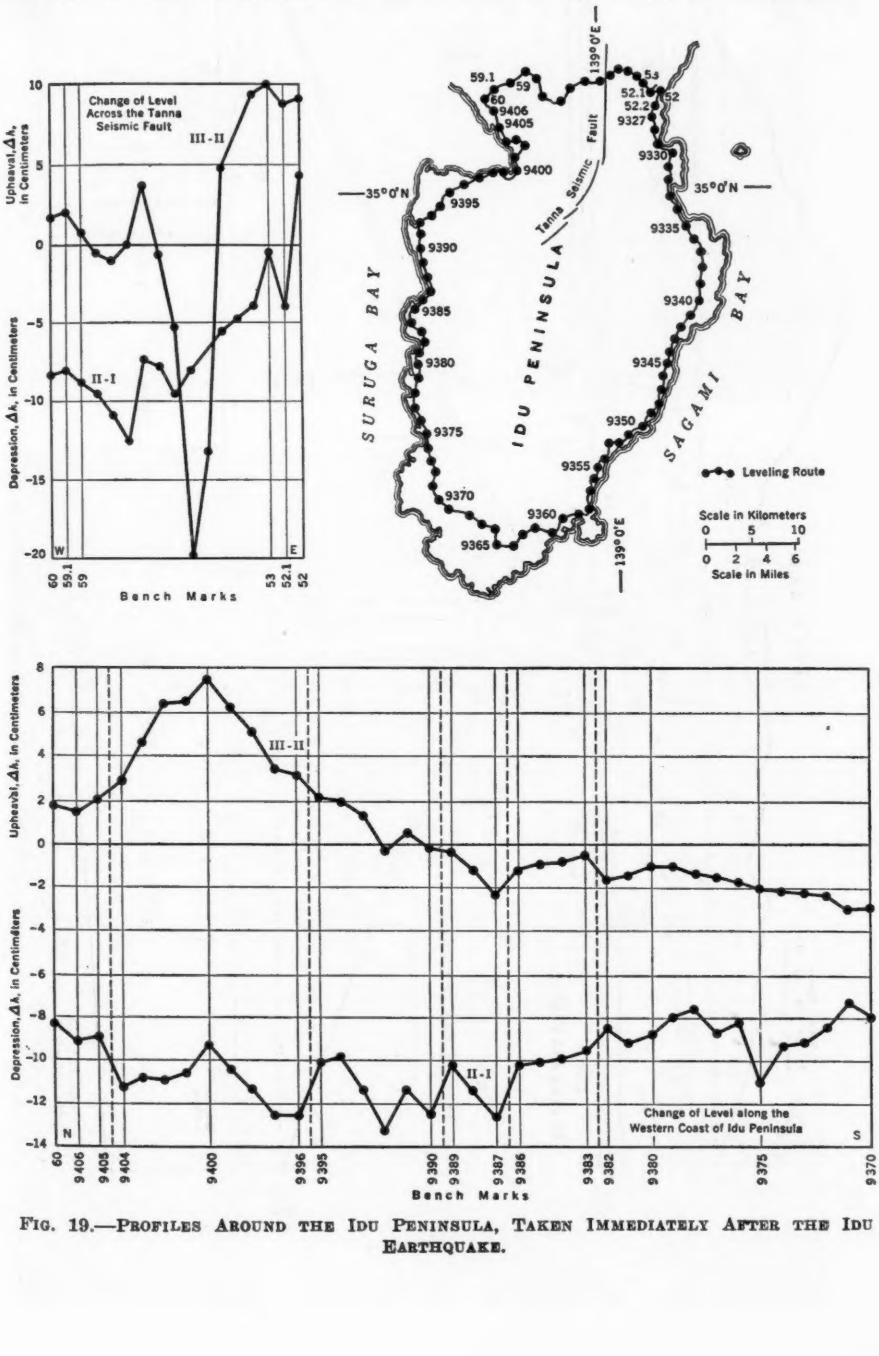

FIG. 19.—PROFILES AROUND THE IDU PENINSULA, TAKEN IMMEDIATELY AFTER THE IDU EARTHQUAKE.

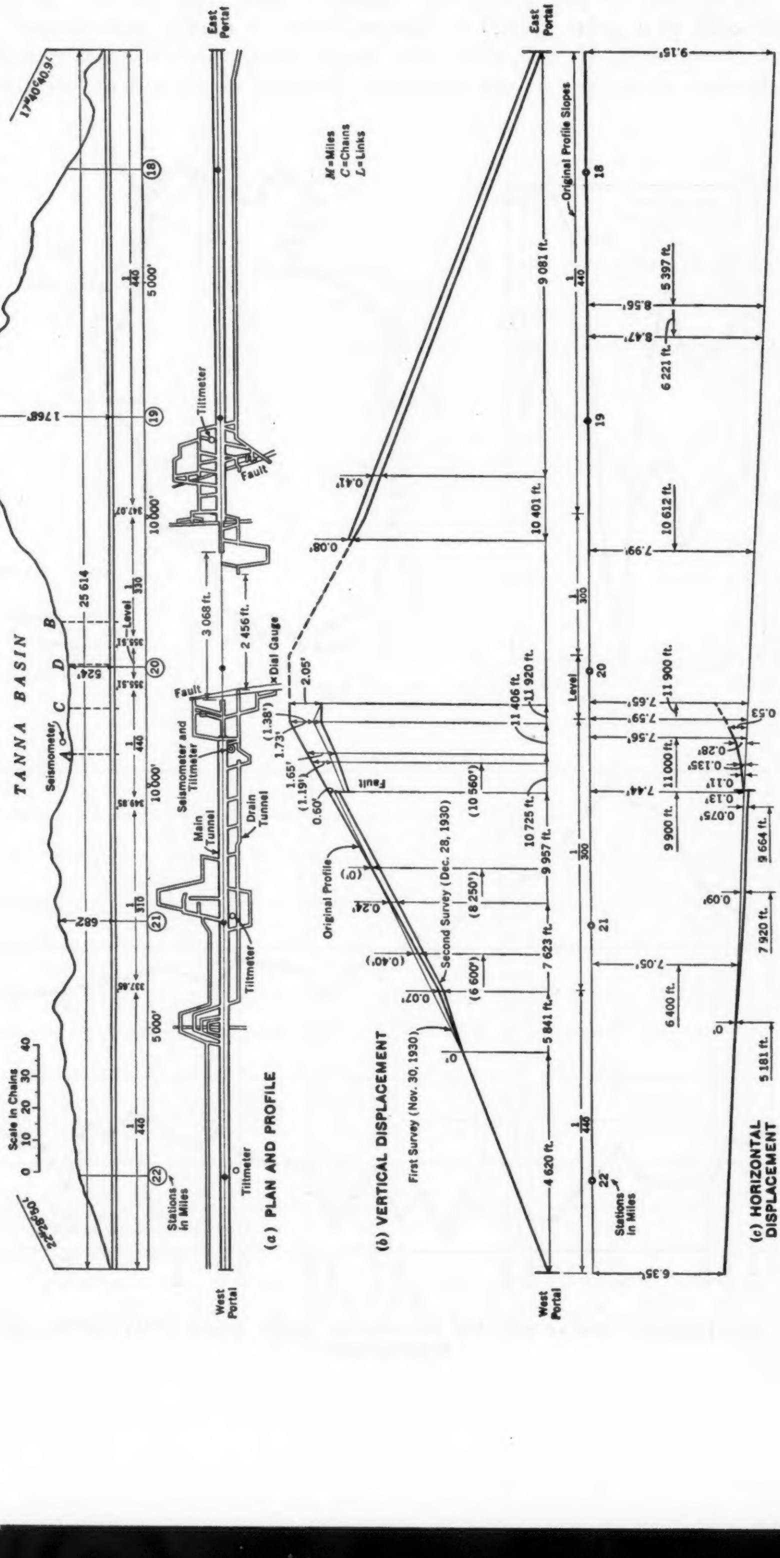

FIG. 20.—EFFECT OF THE IDU EARTHQUAKE ON THE TANNA TUNNEL.

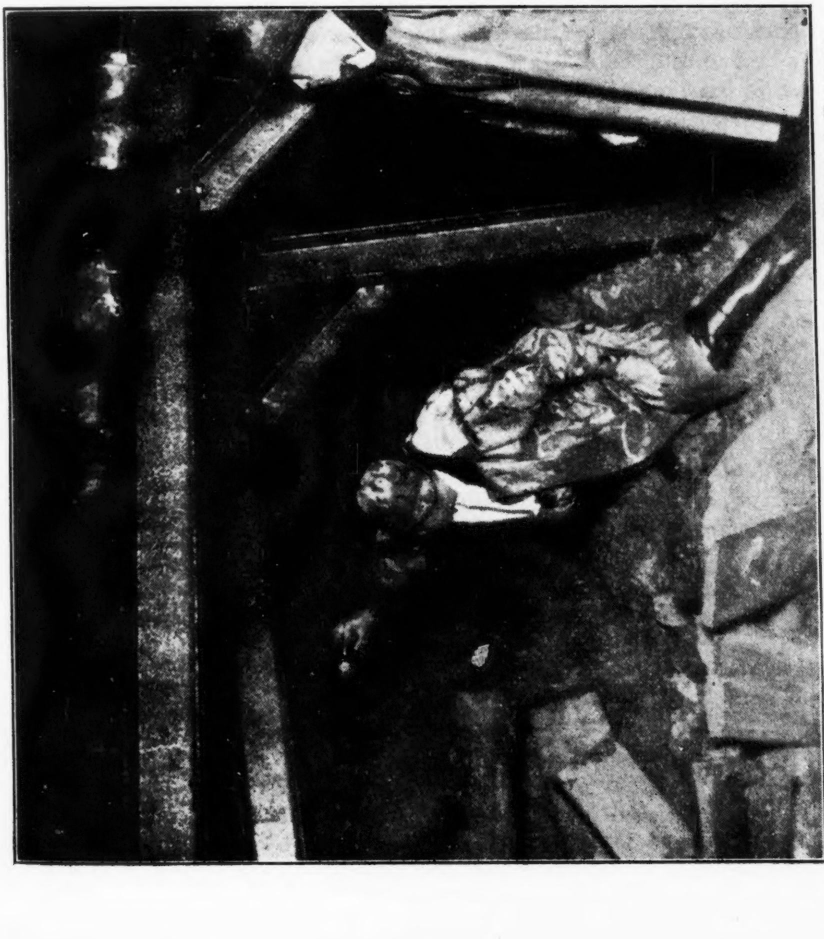

FIG. 21.—FAULT PLANE THAT APPEARED IN ONE OF THE TANNA DRAINAGE TUNNELS AFTER THE IDU EARTHQUAKE.

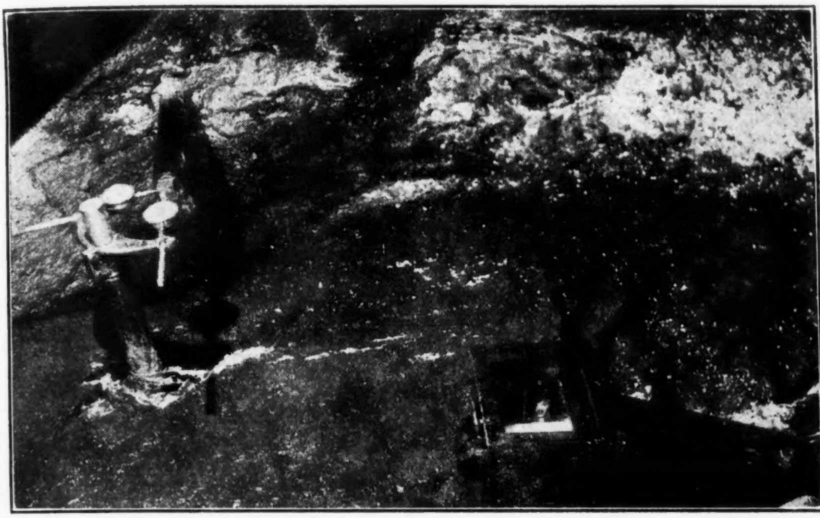

FIG. 22.—DIAL GAUGE FITTED TO STEEL RODS ON EACH END OF A FAULT PLANE IN THE TANNA TUNNEL.

or polished surface, due to dislocation at the fault plane, appeared, showing a relative horizontal displacement of nearly 8 ft. (See Fig. 21.) To measure the relative motion between the two sides of the fault plane, a stout steel rod was embedded in each side and the relative motion was measured by dial gauges fitted between the two rods. (See Fig. 22.) The observations that were begun on December 28, 1930, are still being continued (December, 1931). Although the observations were interrupted at the beginning (when our instrument was disturbed by the installation of another type of instrument by others), a general idea of the relative movement can be gained from Fig. 23. It is seen that in the earlier stage, the displacement had both northern and downward components, but the former component gradually decreased until now it can no longer be detected. The latter component also has diminished gradually, but at present (September, 1931), a displacement of 0.001 mm. per day is observable.

The deformation of the tunnel was surveyed twice by the engineers of the Railroad Department. The result, which is self-explanatory, is shown in Fig. 20. In addition to the aforementioned instruments, Ishimoto tiltgraphs have been installed in four places in the tunnel, for the purpose of measuring continuously the changes of level. The changes of level throughout its entire length were measured at intervals by Mr. R. Takahashi, of the Institute. As the changes of level have to be measured in seventy-two places, each 20 m. apart, the work is very laborious. It is accomplished by means of an apparatus designed by Mr. Takahashi himself, which essentially is a long flexible tube filled with water and fitted with micrometer screws at both ends, for measuring the heights of the free water surfaces. Thus far, this survey has been repeated five times, the results of which are shown in Fig. 24. It will be seen that the changes of level become smaller with time, thus showing the gradual settling of the disturbed crust of the earth.

6.—Underground Seismometry.—For seismometrical work, besides the seismographs that have been installed at several places in and around the disturbed area, two seismographs have been installed for the special purpose of comparing the seismic motions above ground and under ground. With this purpose in view, one instrument has been installed in the tunnel itself, about 500 ft. below the ground surface, and the other above ground on the Tanna Basin, in a position directly above the one in the tunnel. Observations with these instruments gave results very useful to geophysicists, but as they have an important bearing on engineering also, I shall refer to them in detail later.

(IV) Concluding Remarks

I have just given a general survey of geophysical research in connection with the investigations of the three destructive earthquakes, either already accomplished by us or taken up as subjects of intensive study. It has been necessary to confine the subject of this discussion to researches on destructive earthquakes. I have only been able to touch here and there upon even this limited subject. Those who are interested in a more detailed discussion should consult the original papers contained in the *Bulletins* of the Institute.

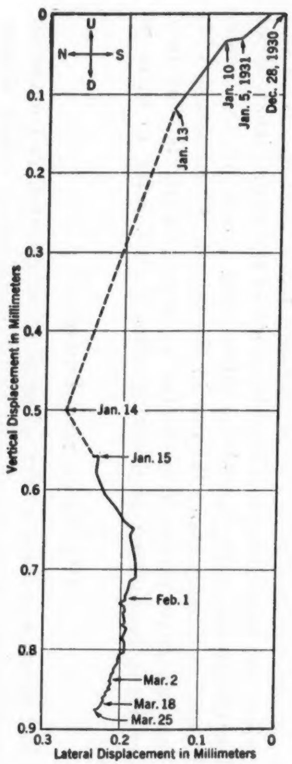

FIG. 23.—RELATIVE MOVEMENT OF STEEL RODS ON OPPOSITE SIDES OF FAULT PLANE IN TANNA TUNNEL.

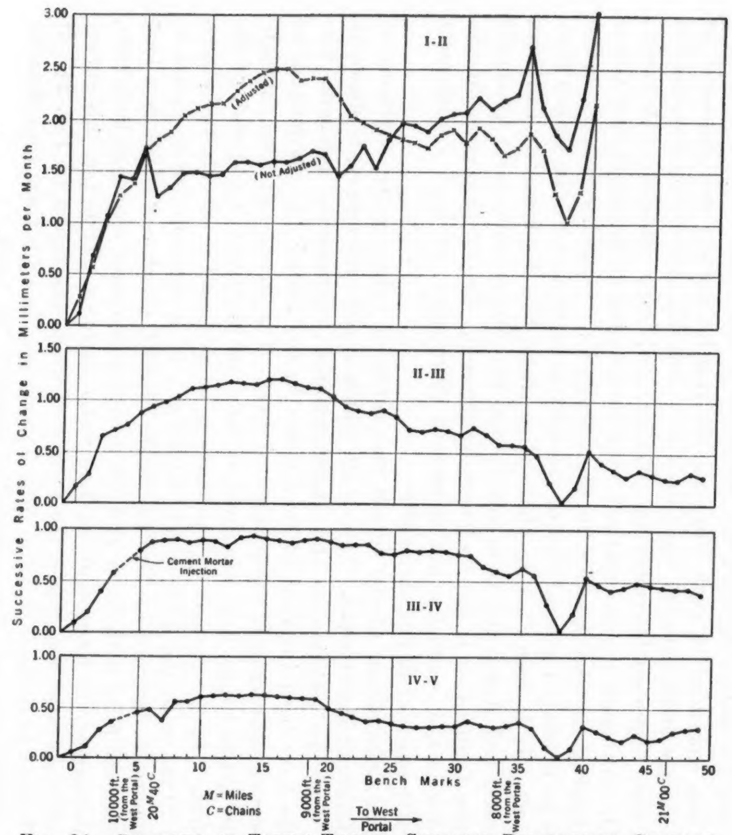

FIG. 24.—PROFILES OF TANNA TUNNEL SHOWING PROGRESSIVE CHANGES IN ELEVATIONS OF BENCH-MARKS AFTER THE IDU EARTHQUAKE.

I wish to take this opportunity to state that our activity is not limited to studies of destructive earthquakes, such as those I have mentioned. Since the establishment of the Institute in 1925, two destructive earthquakes have occurred. These have occupied so much of our attention that we have not had sufficient time to pursue the more fundamental researches. Nevertheless, in the meantime we have not neglected the other kinds of researches that have more direct bearing on urgent problems; nor have we neglected to make preparations for the study of fundamental problems. In addition to researches already mentioned in connection with the great earthquakes, such as seismometry, tilt-measuring, etc., the following work is now in progress, or is being planned for the immediate future:

(*a*) Construction of two underground chambers at depths of 30 and 60 ft., for the study of underground seismic motions.

(*b*) Construction of covered trenches in which to install 20-m. quartz scales, for continuous observations of length changes of the ground.

(*c*) Construction of a permanent seismological station inside the Tanna Tunnel.

(*d*) Continuous measurements of variations in the derivatives of gravity potential.

(*e*) Continuous measurements of the tilting of the ground by means of long water-tubes.

(*f*) Re-survey of heights of bench-marks on the leveling route over the middle part of the main island facing the Pacific.

(*g*) Occasional measurements of the altitudes of some mountains.

(*h*) Occasional precise levels of the City of Tokyo and its environs, and occasional measuring of the geodetic base line.

(*i*) Installation of accelerographs in various places to find the period of acceleration (not the period of motion), of earthquakes inherent to such localities.

(*j*) Concurrently with Project (*i*), observation of the inherent period by means of the seismic motion analyzer.

(*k*) Installation, in suitable localities, of perfect recorders for destructive earthquakes.

(*l*) Installation of strong earthquake recorders in certain suitable office buildings.

(*m*) Installation of strain-meters in certain buildings.

(*n*) Construction of a large shaking-table.

(*o*) Geological survey of Northern Idu.

(*p*) Mathematical investigations.

(*q*) Frequent soundings of Sagami Bay by Langevin's sounding machine. This plan has been proposed to a wealthy man interested in such research, but its realization is not certain.

Thus far, I have referred principally to the work done by our Institute; but that is not the only work done in Japan. The Central Meteorological Observatory and the Geophysical Institutes in the various universities have

accomplished many important investigations, although none of them is interested in the engineering side of seismology, which is the principal subject of this lecture.

For an excellent account of Japanese seismological research, past and present, I recommend the paper by Professor T. Terada, entitled "A Historical Sketch of the Development of Seismology in Japan," and published in "Scientific Japan," a pamphlet distributed to the members who attended the Third Pan-Pacific Congress, held in Tokyo in 1926. In this paper a concise digest is given of all seismological work accomplished in Japan from the time of its first inception until 1926. The work accomplished by our Institute is reported twice each year to Dr. V. Conrad, the Editor of *Gerlands Beiträge zur Geophysik,* and is published in that magazine.

Earthquake Prediction.—Earthquake prediction is naturally very keenly desired by every one living in a seismic country, and it has been the subject of speculation throughout all ages. Even in the present age of scientific development, a destructive earthquake has never been predicted. Instead, we are accustomed to hear after the shaking is over that such and such phenomena had taken place before the occurrence of the earthquake, and that if such phenomena had only been observed beforehand, the prediction might have been possible. It is a pity that in such a case we hear very little of the casual relation of the pre-seismic phenomena to the occurrence of the earthquake, and of whether the occurrence of the earthquake correlated with certainty or merely with some probability to the pre-earthquake phenomena, if correlation did really exist.

Even if earthquake prediction were possible, it must include a complete forecast of the time of occurrence, the place, and the intensity; none of these three items must be lacking. Suppose that a seismologist forecasts that an earthquake of unknown intensity—that is, whether it will overthrow buildings or whether it will be felt only by a tromometer—will take place in a certain locality at a certain time. The result will only be the causing of needless anxiety on the part of the public. In Tokyo, for instance, it is not uncommon to feel the ground shake once or twice in some weeks. For the inhabitants of such a district it is better that they have only incomplete forewarning, without any information as to probable intensity. A vague warning such as is sometimes given by careless seismologists, stating the locality and the probable intensity, but without giving any information as to the time of occurrence, is worse than useless for people living in a seismic country, even if it were a correct guess. In a sense, such a prediction is like stating that we are sure to die sooner or later.

Now let us consider briefly the prospects of earthquake prediction in the light of modern seismology.

(*a*) Meteorological Phenomena.—The occurrence of earthquakes, no doubt, has an intimate connection with meteorological phenomena, such as barometric pressure, its gradient, precipitation, tides, etc. In fact, their intimate correlation with earthquakes has been confirmed statistically by several investigators; but evidently meteorological phenomena merely act as a "trigger" for

starting the earthquake. An unloaded gun cannot be fired by pulling the trigger. Meteorological cause alone is unable to start an earthquake in an unstrained crust or by disturbing the equilibrium of quasi-solid magma. It is very probable that a minor cyclone which passed over the earthquake zone shortly before the occurrence had acted as the "trigger" that precipitated the destructive 1923 earthquake; but cyclones of equal or still greater intensity have passed over the same zone before, without starting an earthquake. Moreover, even if we admit that certain meteorological phenomena do unmistakably bring about an earthquake, the intensity of the induced earthquake may have no relation whatever to the meteorological phenomena. Thus, it seems that meteorological phenomena are not essential data for earthquake prediction.

(*b*) Crustal Movements.—In some quarters it is believed that an earthquake can be predicted from crustal movements, such as the tilting of the ground, the chronic rising or sinking of the land, the changes in the length of the land, and the like; but until we acquire more geophysical knowledge in these respects than we have at present, I cannot support such a view. It is an acknowledged fact that earthquakes are frequent where bradyseismical (slow or gradual) crustal movement is noticeable. We do not know, however, at what stage of a crustal movement of a certain type an earthquake starts in a certain district with a certain intensity at a certain time. Such a problem corresponds to that of the happening of a single one of a great number of events, the occurrence of which is governed by some statistical law yet to be found. To predict the occurence of such an event with certainty is almost hopeless.

According to our observations, the after-shocks of the destructive Tango earthquake and the Ito earthquake were closely correlated with the manner in which the ground tilted. The ground tilts were in turn correlated with the tides; but as already mentioned the manner of correlation is quite different in the two cases. In the case of the Tango earthquake, remarkable after-shocks occurred at the time the tilting of the ground temporarily ceased and began to change its direction (see Fig. 13), while in the case of the Ito earthquake, conspicuous shocks took place while the ground continued to tilt in a certain direction (see Fig. 17). The only condition that was common to both, was the absence of any signs foretelling the intensity of the earthquake that was to come.

Little information is available regarding the linear changes of geodetic base lines. Only one example has been shown (Figs. 8 and 9). If we accept the theory that the peculiar compressional motion observed just before the great earthquake is indicative of changes in the earth's crust that precede a violent convulsion, we should have expected another great earthquake in 1930; but, unfortunately, events have not proved it. Some one interpreted the available data to mean that that particular compressional strain was the harbinger of the Idu earthquake. Even so, the same peculiar compressional strains foretold on one occasion a tremendous earthquake affecting a vast area, and on the next occasion a destructive but local earthquake of different origin. Thus, so far as present knowledge goes, linear ground changes cannot serve as data for the complete prediction of an earthquake.

From the map showing the vertical displacement of the ground in Central Japan (Fig. 4), it will be seen that bradyseismical motion in Japan is very remarkable. If the elevations and depressions of the ground are unmistakable signs of the immediate occurrence of a destructive earthquake, then there is not a piece of land in Japan to-day that is not in imminent danger. Obviously, such is not the case.

In connection with this subject, it is necessary to make a brief mention of the upheaval of the Miura Peninsula, which is alleged to have taken place before the great 1923 earthquake. Common sense, derived from experience with the testing of materials, leads us as engineers to look for some sign preceding the crustal convulsion. It is a pity that we were not successful in obtaining a clue based upon something scientifically sound. As has already been mentioned, the apparent slow upheaval of the sea coast of Aburatubo preceding the great earthquake, as recorded by a mareograph installed there, was carefully analyzed by Professor Terada and Mr. Y. Yamaguchi, who found that the apparent upheaval was attributable to meteorological causes, rather than to any actual upheaval of the land.

It is true that an Omori tiltgraph in the Seismological Institute at the Tokyo Imperial University apparently indicated a gradual change of the inclination of the ground prior to the earthquake; but it must be remembered that the instrument, which is an ordinary long-period horizontal pendulum made of metal with a smoked drum recorder, was not suited for measuring a very small tilt. Moreover, the tilting of the ground where the University is situated is rather easily induced, either directly by precipitation, or indirectly by its cooling effect, and, unfortunately, on the very morning of the day of the great earthquake, the earthquake zone had received a shower followed by intense summer sunshine. The meteorological elements might have strongly influenced the tilting of the ground.

Professor Imamura has concluded, no doubt after taking into consideration the facts just mentioned, that an unusual pre-earthquake tilting actually did take place. Such a pre-earthquake tilting, granted that it did occur, cannot be taken as an indisputable warning of a destructive earthquake, because according to subsequent researches made by Mr. M. Ishimoto, of our Institute, and Mr. M. Tsuji, of the Astronomical Observatory, in which the more reliable Ishimoto tiltgraphs were used, tilts much larger than that claimed to have taken place just before the great earthquake were frequently registered in the Observatory grounds at ordinary times. Furthermore, on June 17, 1931, Tokyo was shaken by a fairly severe earthquake shock with its epicenter very near the Observatory grounds, and Mr. Tsuji found that no unusual tilting was observed either before or after this earthquake. Thus, tiltings of the ground, too, cannot be taken as indisputable signs of earthquakes.

I shall not discuss other phenomena, such as changes of ground-water level, changes in temperature of hot springs, changes in earth electric currents, disturbances of terrestrial magnetism, etc. There is no doubt that they are all in some way or other intimately related to earthquakes, but if they and other geophysical data, such as the form of the geoid and gravita-

tional acceleration, are properly examined, we should be able to throw considerable light on seismology, but it is a regrettable fact that, thus far, we have not been able to find a single indisputable sign by which we can foretell the coming of a destructive earthquake. Notwithstanding the meager appropriation of funds for research, we are doing our best to discover such a sign.

However, as to the possibility of complete prediction, foretelling not only the place, but also the time and the intensity of a destructive earthquake, I am rather pessimistic. While on this subject of earthquake prediction, I wish to acquaint you with what our most prominent geophysicist, Professor Terada, has said in his valuable essay entitled "Prediction of Natural Phenomena."

After mentioning the difference in the macroscopic and microscopic points of view, with special reference to the disposition of crystals in a labile supersaturated solution, he proceeds to the discussion of earthquake prediction. He states that, taking for granted that an earthquake is due to the failure of the elastic equilibrium of the earth's crust, under a certain definite law (actually not yet known), and that the measurement of the strain and other similar elements is possible in every detail, yet even in such a case accurate prediction of the time of occurrence will be impossible, because it resembles the starting of the crystallization of one particular crystal in a supersaturated solution, in that it is governed by some accidental microscopic condition. Needless to say, in this case the macroscopic condition that the solution is supersaturated indicates the possibility of the starting of crystallization as a whole. According to him, in the present state of our knowledge of seismic phenomena, we must rely upon the method of statistics, which ought to be elaborated before arriving at a definite conclusion.

I am of the same opinion as Professor Terada; but although we are unable to predict an earthquake in the strict sense, there seems to be little doubt that some day it will be possible for us to judge whether or not a district as a whole is in immediate danger, and thus to provide against a future destructive earthquake.

It is the duty of seismologists to hasten that day as much as possible. Until such a day comes, and even after it has come, security against earthquakes, in seismic countries, is entirely in the hands of the engineer. My anxiety, therefore, is not so much concerning the inability of seismologists to find Nature's unmistakable warnings of an impending destructive earthquake, as in the present indifference of the majority of architects and engineers to earthquake problems.

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LECTURE II
ENGINEERING SEISMOLOGY

Synopsis

Differences between basic data needed by the engineer and the geophysicist are outlined in this lecture. Ordinary types of seismographs are entirely unsuited for recording the motion of severe earthquakes directly on the site, because they must also be delicate enough to record far distant tremors. Requirements for the "strong-motion" seismograph and accelerograph that offer the best possibilities of supplying useful data to the engineer, are stated.

From observations and studies of the earthquake of 1923 in Tokyo, Dr. Suyehiro concludes that at present the best value to use as a guide to building construction is an acceleration of 0.15 g, although there was abundant evidence to show that still greater accelerations occurred in the epicentral region. Since, even in the most seismic country, however, any building is likely to be subjected to destructive earthquakes only once or twice in its lifetime, the element of economy enters the problem. Despite the fact that the 1923 earthquake was the most severe in the record of seismic history buildings designed on the basis of 0.1 g resisted damage quite well.

The lecture contains illustrative comparisons of simultaneous vibrographs recorded in building frames and in the adjacent ground. A seismic vibration analyzer that works on the principle of selective resonance as in Hartmann's reed frequency meter has been devised by Dr. Suyehiro. The records obtained by his instrument brings out the prevailing natural periods in any locality being studied.

In the design of buildings it is important to consider the relation between the natural period of the structure and that of the ground, the damping effect of the ground, and the mutual action between the ground and the foundation of a building. These problems are virtually impossible to compute mathematically, but one feasible solution seems to be experimentation with models, designed to satisfy the law of mechanical similitude. Experiments with models of wooden buildings are mentioned.

Introduction

Correct seismometric data are equally important for the geophysicist and for the structural engineer; but their needs are not the same. The former, particularly those in Europe, aim to observe the different phases and forms of earthquake waves with the view of locating the position of the origin of the earthquake, to find the velocity of propagation of the seismic waves, and to

study the nature of the different strata forming the earth's crust, etc. For the structural engineer in seismic countries, however, many of such data have little value, although they may interest him indirectly. The duration of preliminary tremors of an earthquake, for example, is of great interest to geophysicists; but structural engineers are generally indifferent to such things. The information urgently needed by the engineer is the intensity and the nature of the principal motions of destructive earthquakes.

The work of seismometric observation is usually in the hands of the geophysicist, who naturally uses seismographs suitable for his object of observation, namely, those instruments having high sensitivity and little stability. It is quite natural, therefore, that whenever a severe earthquake occurs, the recording styluses of all seismographs installed in the strongly shaken districts swing off their recording drums as soon as the principal motion of the earthquake begins, thus at best giving only the record of the preliminary tremor. Indeed, the fact that the recording stylus swings off the drum is generally taken to be evidence of a severe earthquake. Under these circumstances, we have not yet succeeded in obtaining a single complete and reliable diagram of a destructive earthquake, by a seismograph installed near its centrum, to say nothing, for the present, of accelerographs which are more important to engineers than seismographs.

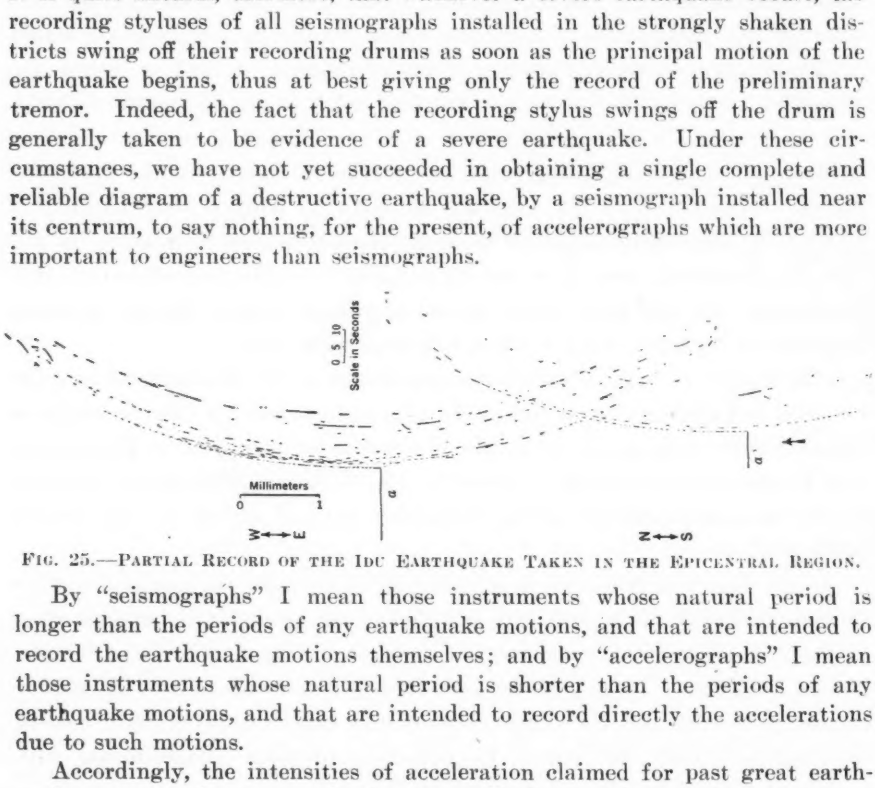


FIG. 25.—PARTIAL RECORD OF THE IDU EARTHQUAKE TAKEN IN THE EPICENTRAL REGION.

By "seismographs" I mean those instruments whose natural period is longer than the periods of any earthquake motions, and that are intended to record the earthquake motions themselves; and by "accelerographs" I mean those instruments whose natural period is shorter than the periods of any earthquake motions, and that are intended to record directly the accelerations due to such motions.

Accordingly, the intensities of acceleration claimed for past great earthquakes are merely guesses, having generally been estimated from the overturning or the displacement of tombstones, and other similar phenomena. Obviously, such methods are too crude, and sometimes even misleading for several reasons: First, the friction of solid bodies when subjected to a complicated vibration cannot be determined; and, second, as a rule, at the very beginning of a destructive earthquake of near-by and shallow origin, the motion starts suddenly. Fig. 25 is a partial record of the recent Idu earthquake to which I have already referred, taken in the epicentral region. Although the apparently abrupt motion at the moment of starting may be

due partly to the sudden release of the recording mechanism of the seismograph from the initial friction, or to the play in the mechanism, yet a more or less jerky start of the seismic motion is scarcely to be doubted.

The impulsive starting of a violent earthquake in the epicentral region was shown very clearly in the Tajima earthquake of 1925. On that occasion, in a village primary school in the epicentral district, copper coins saved by the children were kept in an empty tin can and covered with a lid. It was observed by the school-master that at the moment the earthquake began, the coins threw off the cover and jumped from the tin can, which was still standing. This phenomenon shows that at the beginning of the motion, the vertical acceleration was more than g, the acceleration due to gravity. As a matter of fact, however, the school building (a two-story wooden house), withstood the earthquake well, although it suffered more or less damage.

A motion of an impulsive nature, such as the blow of a hammer, can cause the shifting or the over-turning of small rigid objects, but evidently cannot cause damage to buildings having more or less flexibility. In some cases the intensity of a violent earthquake estimated in the manner referred to may be just such an initial acceleration, which, however, is not the information urgently needed by engineering seismologists.

Therefore, information regarding the intensity of an earthquake must be that obtained in a scientific manner from a reliable record given by a suitable instrument. Indeed, some records were obtained by seismographs in the 1923 Kwanto earthquake, of which I have already given a detailed description, and if those records are trustworthy, they would furnish valuable information, apart from the fundamental question regarding the value of seismograms, which will be considered later.

(I) Strong-Motion Seismographs

Before proceeding to the examination of these records, let us consider briefly the requirements necessary for a strong-motion seismograph, in order to see, on the one hand, if the instruments that registered the violent earthquake had been properly constructed, and, therefore, were serviceable; and, on the other hand, to see what the future design of this kind of instrument ought to be.

(*1*).—*Type.*—Let us first consider the type of the instruments. Among the horizontal seismographs, the horizontal pendulum type introduced by Professor Ewing is most widely used. Although for ordinary use the advantages of his principle in seismometry are scarcely to be doubted, it seems that for measuring an earthquake of unusual strength, there remains much to be desired.

Now let us inquire whether or not an instrument constructed under this principle is suitable for measuring a violent earthquake. As I have reported elsewhere[6], the horizontal pendulum ceases to be a faithful recorder if it is subjected to a strong acceleration in the horizontal direction perpendicular to the motion which the instrument is intended to record.

[6] *Proceedings*, Imperial Academy, 3 (1927), No. 3; see, also, Gutenberg's Handbuch der Geophysik, Vol. IV, 2, p. 387.

Let the free oscillation of a Ewing horizontal pendulum (Fig. 26) be given by,

$$I \frac{d^2\theta}{dt^2} + M g \phi l \theta = 0$$

in which,

I = mass moment of inertia of bob weight (including accessories) about the pivoting axis, AC;
M = mass of bob weight;
l = distance of center of mass from pivoting axis;
ϕ = inclination of pivoting axis to the vertical; and
θ = angle of oscillation of the central plane, CD, about the position of equilibrium, CE.

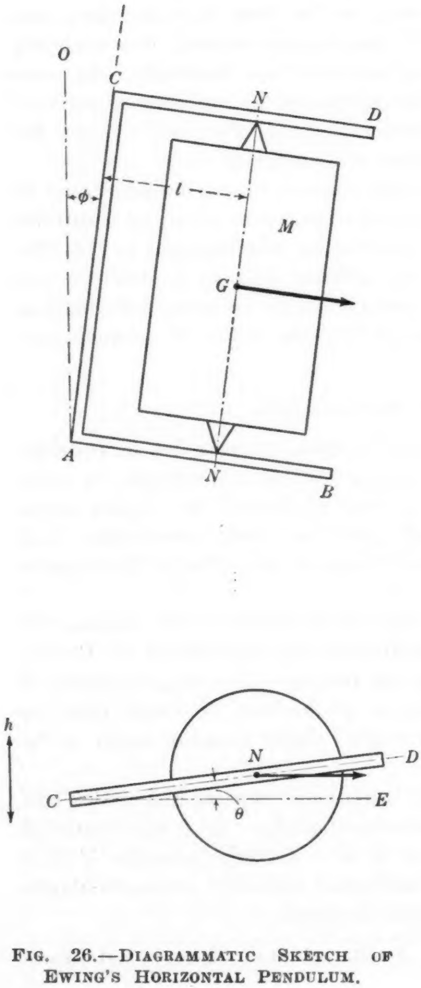


FIG. 26.—DIAGRAMMATIC SKETCH OF EWING'S HORIZONTAL PENDULUM.

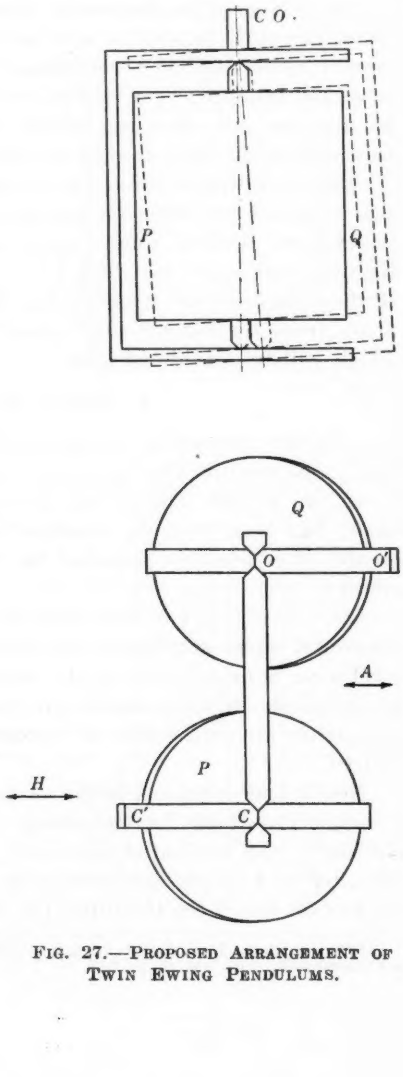


FIG. 27.—PROPOSED ARRANGEMENT OF TWIN EWING PENDULUMS.

When the instrument is subjected to a harmonic oscillation (of maximum acceleration, α, and of circular frequency, p) in the longitudinal direction (H), horizontally, as stated, it can be shown that the motion of the pendulum is given by the equation,

$$\frac{d^2\theta}{d\tau^2} + \left\{ p_0 + p_1 \left(l^{2i\tau} + e^{-2i\tau} \right) \right\} \theta = 0 \quad \ldots\ldots\ldots\ldots\ldots (2)$$

in which,

$$\tau = \frac{p}{2} t;\ p_0 = \frac{\dfrac{M g \phi l}{I}}{\left(\dfrac{p}{2}\right)^2};\ p_1 = \frac{1}{2} \frac{\dfrac{M \alpha l}{I}}{\left(\dfrac{p}{2}\right)^2};\ \text{and } i = \sqrt{-1}.$$

Equation (2) is a particular form of Hill's equation, named after G. W. Hill, an eminent American astronomer. As the equation is very important to astronomy, thorough investigations have been made of it, and it has been described in several textbooks dealing with advanced differential equations; therefore, it is not necessary to recapitulate them herein. It may be mentioned, however, that a deduction from the differential Equation (2) shows that when the longitudinal acceleration, α, is large, the horizontal pendulum loses its stability, this effect being especially conspicuous when the frequency of the longitudinal motion is one-half the natural frequency of the instrument. A record under such circumstances is obviously untrustworthy.

This behavior of the Ewing horizontal pendulum need not be specially considered in the case of an ordinary seismograph for measuring distant earthquakes, but in constructing a strong-motion seismograph it is very important.

To eliminate this undesirable feature of the Ewing horizontal pendulum, the simplest way is to make its natural period of oscillation large, retaining its stability to a reasonable extent. The only way to attain these two counteracting requirements simultaneously in a simple manner is evidently to make the size of the instrument sufficiently large. Another less simple but practicable method which I propose is to arrange a pair of Ewing horizontal pendulums of the same construction side by side, in opposite directions and parallel to each other, the bracket frames holding the bobs being united by a coupler link over the axis holding the weights (Fig. 27). When such an arrangement is subjected to an accelerating motion in the longitudinal direction, obviously, the instability, if any, of one of the pendulums is compensated by the increased stability of the other, and the instrument is thereby prevented from making an undesirable motion.

(2).—*The Length of the Boom.*—For ordinary seismographs, the length of the arm is immaterial; but for a strong-motion seismograph this matter must not be overlooked. It will be seen that when a seismograph with a short arm is subjected to a vibration having a large amplitude, its behavior can no longer be inferred from the theory of forced harmonic motion, as it then belongs in the category of forced pseudo-harmonic motion. No mathematics are needed to see that, if the amplitude of an earthquake is comparable to the length of the arm, the insensibility of the bob to the external motion can scarcely be expected.

For this reason, the arm must be sufficiently long; but it must be borne in mind that the longer the arm the more sensitive is the instrument to the tilting of the ground, so that too much length is not desirable; 1 m., or so, seems to be the proper length.

Besides these fundamental requirements, there are several minor but no less important points advisable in the construction of a strong-motion seismograph, as follows:

(1) The instrument should be capable of recording a maximum amplitude of vibration of ground equal at least to 30 cm.

(2) Every part of the apparatus should have ample strength and stiffness.

(3) The record of motion should not be multiplied, but on the contrary it should be reduced, say, to one-half.

(4) The bob should be sufficiently heavy, so as to prevent it from being dragged by the friction of the recording point and other parts.

(5) The natural period of the pendulum should be sufficiently long; but if it is obtained only by greatly lessened stability, the period to a certain extent, should be sacrificed to stability.

(6) The construction of the axis of suspension should receive careful attention so that the connection does not give way under a strong impulse. The use of ball-bearings immersed in lubricating oil is advisable.

(7) The recording drum should have an ample margin of length at both ends, so as to insure a perfect recording of an earthquake, even when it occurs at the beginning or at the end of a record.

(8) A delicate damper often becomes a cause of trouble.

(9) The use of clock-work for driving the recording drum is not advisable, but if its use is unavoidable (for instance, for seismographs to be used in country districts), the speed governing the rotation should be regulated by an eddy current brake, or by a suitable governor, but not by an air-brake.

(10) To obtain a diagram with open time scale (as distinguished from condensed scale), the circumferential velocity of the recording drum should be as high as possible (say, at least 12 cm. per min.). For this purpose alone, the use of an electric motor having a uniform rotation for driving the drum is highly recommended. Needless to say, in this case the electric current should not be taken from the municipal supply.

(11) The trigger arrangement for starting the recording drum is not convenient for estimating the period of motion, owing to the accelerating speed of the drum at the beginning. If it is used, the recording stylus or the beam of light (but not the drum), should be controlled by the trigger. I had two bitter experiences with the trigger arrangement. On a certain midnight, a slight fore-shock preceding the main disturbance started the recording arrangement (described in Lecture III) before it could record the main shock. On another midnight, a distant earthquake caused the same premature starting, so that no record was taken of the main disturbance.

(12) The time should be marked independently of recording points. Simultaneous use of the recording point as a time marker is objectionable.

(13) The bed-plate of the instrument should be strongly attached to the ground by studs or by other means.

(14) The instruments should be well protected from injury by falling débris.

In the foregoing I have merely enumerated precautions and suggestions so far as my knowledge permits, from past experience. I am, therefore, afraid that other equally important matters have not been mentioned.

(II) Intensity of the Destructive 1923 Earthquake

Having described the necessary requirements for a strong-motion seismograph, let us now revert to the question of the intensity of the 1923 earthquake.

True, some records of this earthquake have been taken, as previously mentioned, but none of them was successful, being either incomplete or unreliable. Moreover, it is regrettable that at that time we had neither the seismographs satisfying the necessary conditions just enumerated, nor any accelerographs. For the purpose of reference, however, I will show some diagrams obtained by seismographs. One of them (Fig. 28) gave a comparatively continuous record, but was nevertheless imperfect.

The record in question was obtained by a seismograph, the particulars of which are, as follows:

Type	Ewing's horizontal pendulum.
Length of arm.............	20 cm.
Weight of bob..............	2.1 kg.
Natural period of pendulum..	10 sec.
Drum	Driven by clockwork controlled by an escapement.
Damper	Vane immersed in oil bath.
Pointer magnification	Two times.
Peripheral speed of recording drum	4 cm. per min.

From what I have said before, it is seen that in many respects this instrument was not suitable for registering a violent earthquake, especially one like that of 1923, in which the main motions had enormous amplitudes and long periods.

However, Professor Imamura, after a careful examination of this record, concluded that at the beginning of the principal motions (marked *fg* in Fig. 28), the full amplitude was about 9 cm. and the period about 1.3 sec., for which the computed acceleration proves to be about one-tenth of the acceleration due to gravity. It is a pity, however, that not only did the point of the stylus for the north-and-south (NS) component run off after a few oscillations of the principal motion, but the record of the other component shows that the motion of the earthquake exceeded the maximum limit of amplitude allowed for recording. No doubt the record is very valuable, yet it failed to give us information urgently needed by engineers, aside from the more basic question of the seismograph records themselves.

However, as the preliminary portion was perfectly recorded, this record is invaluable to geophysicists. Indeed, it was by means of this record that

within half an hour of the occurrence of the great earthquake, Professor Imamura located its origin with confidence and explained the nature of the earthquake to the general public.

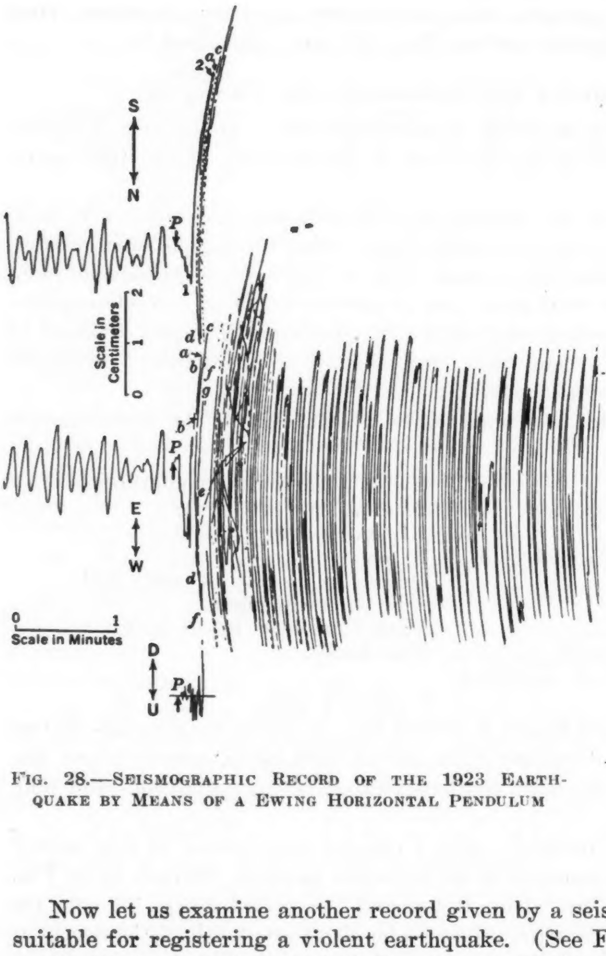


FIG. 28.—SEISMOGRAPHIC RECORD OF THE 1923 EARTHQUAKE BY MEANS OF A EWING HORIZONTAL PENDULUM

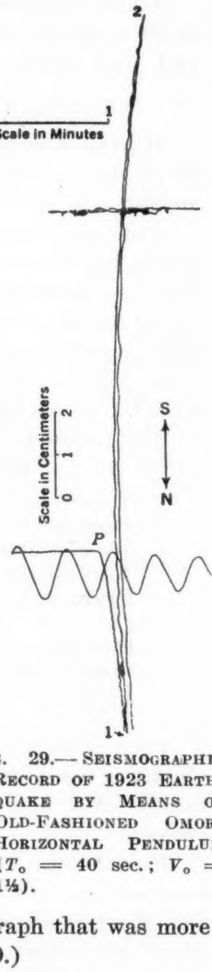


FIG. 29.—SEISMOGRAPHIC RECORD OF 1923 EARTHQUAKE BY MEANS OF OLD-FASHIONED OMORI HORIZONTAL PENDULUM (T_0 = 40 sec.; V_0 = 1½).

Now let us examine another record given by a seismograph that was more suitable for registering a violent earthquake. (See Fig 29.)

The instrument had the following characteristics and constants:

Type	Omori's horizontal pendulum.
Length of boom	1 m.
Weight of bob	15 kg.
Natural period of pendulum	40 sec.
Drum	Driven by clockwork controlled by an air-brake.
Damper	None.
Magnification.	1.5 times.
Peripheral speed of drum	4 cm. per min.

It will be seen that this instrument was more suited to registering a violent shock than the one previously described. In fact, successful recordings of several other severe earthquakes in the past proved its competency; but, unfortunately, the pointer went off the drum at the beginning of the principal motion. It is, however, worth special mention that, according to the record, even at the first motion of the earthquake, the amplitude of one component of the motion was about 17 cm. (the possible tilting of the ground might have affected the record to a certain extent. However, as the period of the instrument was very long, the effect due to the tilting on the record, of comparatively quick earth vibrations, was probably not conspicuous). The accelera-

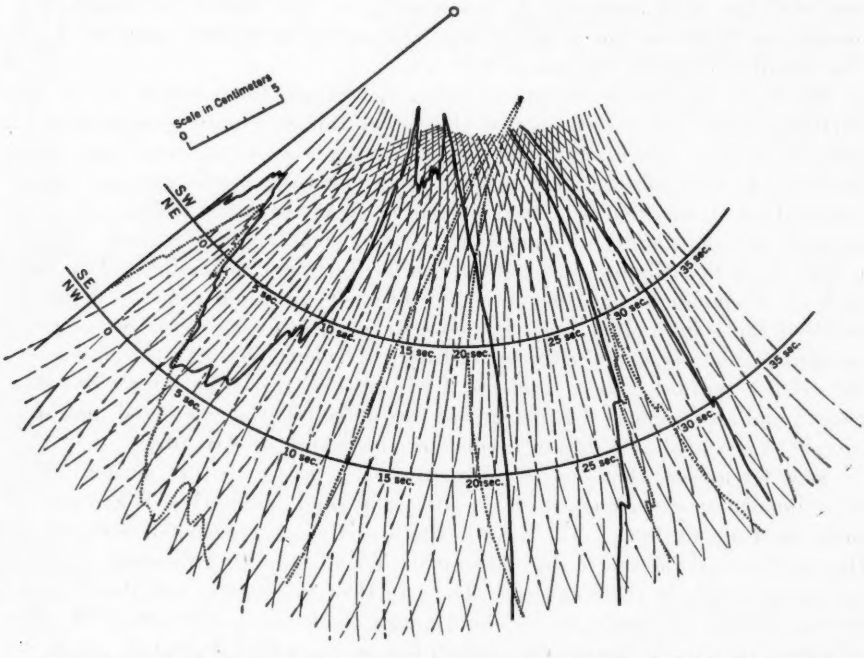


FIG. 30.—SEISMOGRAPHIC RECORD OF THE 1923 EARTHQUAKE BY MEANS OF AN OLD-FASHIONED EWING SEISMOGRAPH.

tion of this motion as calculated was not large, being only about 15 gal. (15 cm. per sec.^{2}) or so, estimating the period at 4.9 sec. and assuming that the motion was simple harmonic. (This assumption is open to question, although it is generally accepted.) The successive waves, if my memory serves right, had probably still larger amplitudes. At first thought one might question the likelihood of such a large motion occurring; but another record (Fig. 30) obtained with an old-fashioned and less reliable Ewing seismograph with a disk recorder, practically confirmed the large amplitude. Moreover, the fact that the full amplitude (as registered in Tokyo) of the Idu earthquake of 1930 was 15 cm., strengthens the conclusion that the occurrence of such a large amplitude as that mentioned is not impossible.

With regard to the record given by the Ewing seismograph, it was practically continuous from beginning to end, except that at intervals the pointers went off the recording disk and thus interrupted the record. Although the instrument was originally intended for use as a strong-motion seismograph, it had several drawbacks. The most serious was that, notwithstanding the defective mechanism for driving the recording disk, it was not provided with a time marker. Moreover, the natural periods of the pendulums were comparatively short, being only about 7 sec. Therefore, the acceleration computed from it is not trustworthy. Disregarding these defects, however, by roughly calculating in the usual way, by means of the amplitude and the period of the main motion, and by assuming that the motion was simple harmonic, we find that the resultant acceleration of the motion marked, A, on the record is roughly 120 gal., or 0.12 g.

However, as will be discussed later, the maximum acceleration of the earthquake motion and the period that gives this maximum acceleration do not, as a rule—especially in a severe earthquake—arise from the main motion. Instead of that, they arise from secondary motions that are superimposed on it, or from that part of the cycle of the main motion which is actually not harmonic. To measure exactly such an acceleration and its period from an ordinary seismogram is difficult, if not impossible. This fact in itself is enough to convince us of the necessity of using properly designed accelerographs in engineering seismometry; needless to say, improper accelerographs are nearly as valueless as ordinary seismographs. As this is one of the most important of our problems, I will deal with it fully later. For this reason, and also because the instrument was defective, the calculated intensity cannot be accepted as a datum needed for the design of structures.

Now let us see what our past experience has to say in the matter. The most important experience in the past is the semi-destructive Tokyo earthquake on June 20, 1894. The record of this earthquake was taken perfectly at Hongo, situated on the higher section of Tokyo, and its principal motions were very simple in their nature.[7] The late Professor Omori calculated from the record that the maximum acceleration up town was nearly 0.05 g. He also estimated that in the lower town, which lies on low alluvial ground, the maximum acceleration was about 0.1 g. This latter value was perhaps less correct than the former, because it was deduced merely from the over-turning of columns and tombstones.

As will be seen later, however, it has been observed that, in general, the intensity of earthquakes down town is 1.5 to 2.0 times that of those up town. This fact does not seem to disprove Professor Omori's estimation. Now, the damage to buildings caused by the great 1923 earthquake up town was somewhat more severe than that caused down town by the semi-destructive 1894 earthquake. From this fact, too, it can be inferred that the maximum acceleration of the great earthquake up town was of the order of about 0.15 g. Moreover, according to my experience at Hongo, at the moment of greatest severity of the shock, I felt the intensity to be nearly the same as what one

[7] See Milne's "Seismology," p. 138.

feels in a motor car driven at moderate speed on a poorly paved road, or as that in a street car, the average acceleration of which, according to the measurement with an accelerograph by Professor Ishimoto and Mr. Nasu, is between 0.15 g. and 0.20 g.

Thus, unfortunately, no recording apparatus gave a reliable record of the great earthquake of 1923, so that the correct value of its maximum acceleration is unknown; but it seems that the estimated value gives an idea of the order of magnitude, and we cannot but adopt it, namely, 0.15 g, or a little larger, as a datum for guiding earthquake-resisting structural design, keeping the fact in mind at the same time that the part of Tokyo in which the earthquake motion was observed was not the most affected. There is abundant evidence to show that the motion had a still greater acceleration in the epicentral region.

What is more regrettable is the fact that absolutely no information as to the intensity of the great earthquake was obtained for the down-town district where comparatively high modern masonry buildings were standing. Every effort was made, however, by Mr. Nasu of our Institute (then assistant to Professor Imamura), to compare the intensities of one and the same earthquake in the up-town (mostly diluvial) and in the down-town (mostly alluvial) districts of Tokyo by measuring simultaneously the after-shocks of the great earthquake at various points in the capital. The observations revealed the fact that neither the periods nor the amplitudes in these two different districts had any definite ratio, but, generally speaking, the computed accelerations were approximately 50 to 100% stronger in the down-town than in the up-town districts.[8] The late Professor Omori's observations on the semi-destructive Tokyo earthquakes of December 8, 1921, and January 14, 1923, showed similar results, the observed data being as in Table 1.

TABLE 1.—OBSERVATIONS BY F. OMORI ON THE SEMI-DESTRUCTIVE TOKYO EARTHQUAKES OF DECEMBER 8, 1921, AND JANUARY 14, 1923

Name of place	East and West Component				North and South Component			
	Double amplitude, in centimeters	Period, in seconds	Accelerations, in centimeters per second[2]	Ratio of acceleration	Double amplitude, in centimeters	Period, in seconds	Accelerations, in centimeters per second[2]	Ratio of acceleration
Earthquake of December 8, 1921								
Hongo (up town)	5.0	3.6	8	1	3.0	1.7	20	1
Hitotsubashi (down town)	6.6	3.2	13	1.6	3.5	1.6	27	1.4
Earthquake of January 14, 1923								
Hongo	1.3	1.4	13	1	1.4	1.3	16	1
Hitotsubashi	1.8	1.3	21	1.6	1.4	0.9	34	2.4

With regard to these data, it must be said that, for reasons which will be given later, the intensity of acceleration computed from a seismogram is rarely correct, so that the ratios of the intensities of acceleration just men-

[8] Rept., Imperial Earthquake Investigation Committee, No. 100A, 1925.

tioned should be understood as being merely qualitative. According to Professor Ishimoto's most recent investigation of this problem with accelerographs, the ratio of the intensity of acceleration on low ground to that on high ground seems to depend upon the nature of the earthquake; for instance, in earthquakes of slow motion the ratio varies between 1.5 and 3, while in those of quick motion the ratio is unity or slightly larger. Thus, it can be accepted as established that the intensity of ordinary earthquakes is larger down town than up town, but I cannot supply the correct ratio of the intensities in the case of the great 1923 earthquake. The ratio might not have been very different from that in ordinary earthquakes.

The distribution of collapsed wooden houses in the great earthquake showed that the ratio of its intensities in these two sections of the city also followed the same rule. The intensity was far greater in the low ground than in the up-town districts, which are situated on diluvial ground.

Thus, there is every reason to believe that the acceleration of the great earthquake on the low alluvial ground of Tokyo reached the enormous intensity of, say 0.2 g, or more (taking a conservative value, say, 1.5, as the ratio of intensities just stated). Therefore, remembering that the capital is situated some distance from the epicentral region, it will not be improper to assume that the acceleration in the worst locality along the coast of Sagami Bay, which is believed to be the central area of the earthquake, was more than 0.3 g.

(III) Seismic Factor and the Lack of Seismic Data

To build a structure to withstand such a large acceleration as 0.3 g, or greater, and to provide it with a sufficient margin of strength, is evidently a matter of the utmost difficulty, if it is possible at all. As a matter of fact, however, a building even in the most seismic country is likely to be subjected to destructive earthquakes only once, or at the most twice, in its life time. Therefore, too ample strength means only extravagance. Thus, remembering that Tokyo was not the worst locality, it seems to me that a building properly designed to provide against a horizontal acceleration of, say, 0.15 of the gravitational acceleration, with a reasonable factor of safety, would safely withstand destructive earthquakes of the intensity of that of 1923, which is one of the most severe in the record of seismic history. Indeed, our own building code provides for a horizontal acceleration of 0.1 g, but some engineers are not satisfied with this seismic factor and voluntarily increase it.

However, owing to lack of reliable information on the intensity of destructive earthquakes, I cannot make positive statements. Until we are informed of the intensity and the period of destructive earthquakes that have been measured in a scientific manner, we can design an earthquake-resisting building only under assumptions based on such rather unsound premises.

As a practical problem, however, the actual fact that buildings designed on the 0.1 g basis (or thereabouts), resisted this earthquake fairly well is a datum more valuable than any other argument. In an engineering design there is nothing more important than practical data. In naval architecture we design a sea-going ship under the assumption that the maximum bending

moment to which it will be subjected is when it rides on a "standard wave" (a trochoidal wave having the length equal to the ship's length and the height one-twentieth of the length) without knowing much about the actual seas which it will possibly encounter. Long experience justifies such an assumption, provided we judge the "working stress" properly.

If such a practical method of designing a ship is successful, why can we not follow the same procedure in the design of earthquake-proof structures? Needless to say, the number of data are incomparably abundant in naval architecture. It is very likely that even at this moment ships in some part of the ocean are struggling with a heavy gale and their strengths are being tested.

Happily for human beings and unfortunately for seismologists, earthquakes do not occur as often as the launching of ships, and, therefore, our data are very scant. The most modern buildings, built of reinforced concrete or of steel framing, have so far been "tried at the bar" of the most severe earthquake only twice in the history of seismology—once in San Francisco, Calif., and once in Tokyo; but most unfortunately for the science, in both cases nothing more than flimsy evidences of the intensity of the earthquake have been left to us. Therefore, we cannot but avail ourselves of practical data obtained in these two earthquakes.

In any case, more information is urgently needed. Engineering seismologists must prepare suitable strong-motion seismometers and accelerographs, and after distributing them in the seismic regions, await with patience the useful data that must come in the future. If, however, circumstances do not permit the installation of both these instruments at the same place, then the latter should be preferred to the former. Moreover, these instruments should be placed on the ground where important buildings are standing, and not on specially selected firm ground, as is generally done.

In Japan, since the occurrence of the 1923 earthquake, the need of severe earthquake recorders has keenly been felt in seismological circles. At present, not only our Institute, but also some of the principal observatories are provided with seismographs intended to record severe earthquakes; but it is to be regretted that none of the instruments is equipped in all respects for the purpose intended. For instance, in the recent Idu earthquake, a strong-motion seismograph that was installed in the epicentral region failed for the reason that not only was the magnification excessive (two times), but also because the maximum amplitude allowed for was too limited.

This failure for the second time, stimulated us to take up the matter more seriously, and Dr. T. Okada, the Director of the Central Meteorological Observatory, intends to distribute more perfect strong-motion seismographs among his principal observing stations. As for me, our Institute is manufacturing not only strong-motion seismographs, but also accelerographs, to meet the necessary requirements just enumerated. In the present state of the development of seismology, we cannot yet foretell where the next great earthquake will occur, so we are unable to select the proper localities for their installation. If, however, a great earthquake were to occur in a district where such seismographs and accelerographs are installed, I expect with confidence that we shall not repeat the failure.

(IV) Accelerographs

Thus far I have dealt principally with the seismic motion recorder; but, as I have remarked, according to our experience, an accelerograph for recording directly the acceleration of an earthquake is more important for us; because seismic waves, especially those of an earthquake of near-by origin, are far from being of the simple harmonic type, so that the acceleration computed by the ordinary formula, namely, amplitude $\times \left(\frac{2\pi}{\text{period}}\right)^2$, has little physical meaning, and, moreover, in some cases, the form of the wave is so choppy that even the determination of the amplitudes and the periods themselves is impossible. It is also to be remembered that the maximum acceleration is frequently given by the secondary motions, with comparatively small amplitude but short period, which are apt to be masked by the main motion. The necessity of an accelerograph in such a case cannot be too strongly emphasized.

As the principle of the accelerograph is described in some of the modern textbooks on seismology, it is not necessary to give a description here; but it may not be without interest to you to see some results of the comparison of the intensity of the maximum acceleration of earthquakes given directly by an accelerograph with that computed from seismograph records (see Table 2).

TABLE 2.—Comparison of Intensities of Maximum Acceleration by Seismograph and Accelerograph

Tokyo earthquake	Maximum Motions Given by Seismographs				Record of an Accelerograph	
	Full amplitude, in centimeters	Period, in seconds	Computed acceleration, in gal.	Speed of drum	Full amplitude, in centimeters	Acceleration, in gal.
August 20, 1930......	0.042 (4S–5N)	0.30	9.2	Slow	4.14	5.6
The same earthquake.	0.064 (4S–5N)	0.46	5.9	Rapid	....	....

The accelerograph used is that designed by Professor Ishimoto, one of our colleagues, particulars of which are as follows:

Type	Inverted pendulum, and optical recording.
Weight of bob..............	3.2 kg.
Natural period of oscillation without damping	0.08 sec.
Attenuation value $(\frac{K}{2})$ of the damped oscillation	Nearly 30.
Optical magnification of motion of center of bob.......	1 700 times.

One centimeter of the amplitude of record corresponds to 2.7 cm. per sec.^{2} This instrument has subsequently been remodeled to be used for engineering researches. For that purpose the magnification was reduced for recording mechanically, and the natural period of oscillation of the bob was increased properly, the particulars being as follows:

Natural period of oscillation without damping	0.15 sec.
Damping	Nearly critical.
Indication constant	1 cm. = roughly, 10 cm. per sec^2.

An instrument for recording severe earthquakes is now under construction, in which the indication constant is 1 cm. = 50 cm. per sec^2.

For the purpose of reference, the records of the earthquake mentioned in Table 2, taken with the Ishimoto optical accelerograph, an ordinary seismograph, and a seismograph of the same type but with a rapidly revolving

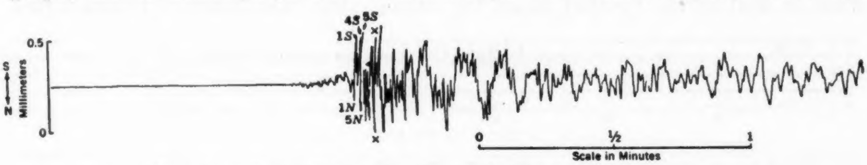


FIG. 31.—CURVE RECORDED BY AN ORDINARY SEISMOGRAPH

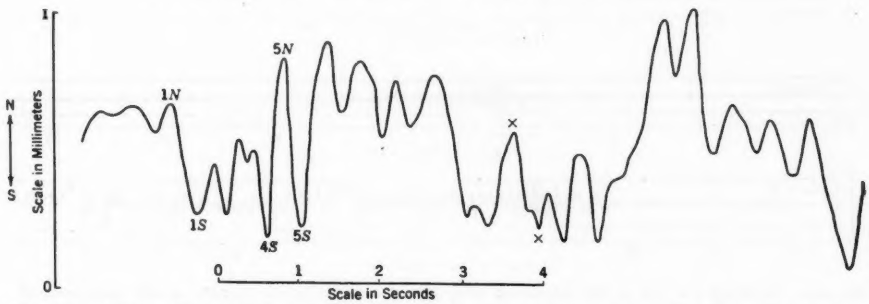


FIG. 32.—CURVE RECORDED BY AN ORDINARY SEISMOGRAPH WITH A RAPIDLY REVOLVING RECORDING DRUM

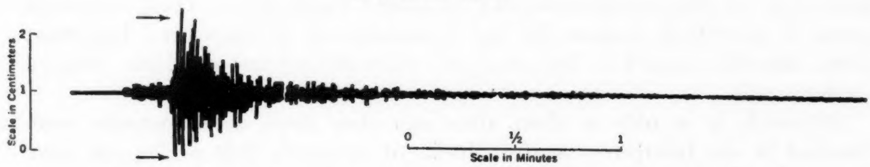


FIG. 33.—CURVE RECORDED BY AN ISHIMOTO OPTICAL ACCELEROGRAPH

recording drum, are shown in Figs. 31, 32, and 33. The computed values of acceleration in Table 2 are those of the motion marked 4S–5N in Figs. 31 and 32. It is seen that the value computed from the diagram with the condensed time scale (the upper row) is quite erroneous, while that obtained from the one with open time scale (the middle row) is less so. This fact also shows that a quick-running recording drum in a seismograph is desirable, if it is to be used for obtaining engineering data.

Even with such a seismograph, however, we cannot realize our object in some cases. In the foregoing I have selected, as an illustration, the record of an earthquake in which seismic waves were composed of comparatively regular trains of nearly harmonic motions, and, therefore, the ordinary method of computing the acceleration could be applied without much error. It will be remembered that even in such an exceptionally favorable case, an ordinary close diagram (with a recording speed of, say, less than 6 cm. per sec.) is apt to lead to a false estimate. As another example, I will show, side by side, a seismogram of an earthquake and its accelerogram taken by Ishimoto's mechanical accelerograph (Fig. 34). This earthquake showed the characteristics so commonly observed in that the principal motions had large amplitudes, but comparatively long period, and, on which motions, secondary motions were superposed. It is evident that an accurate estimation of acceleration as well as the "period of accelerations" (the time interval between two

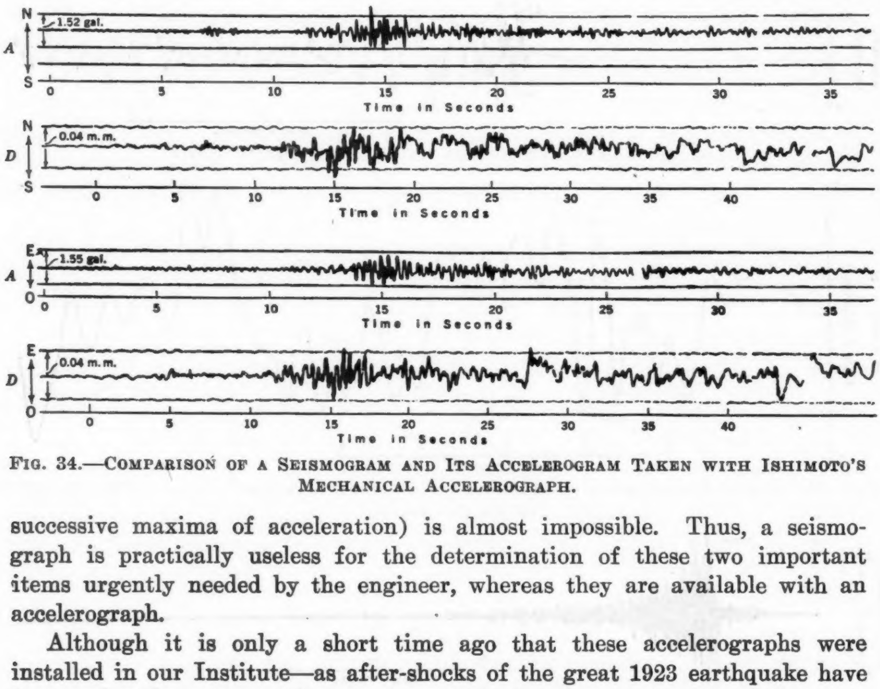


FIG. 34.—COMPARISON OF A SEISMOGRAM AND ITS ACCELEROGRAM TAKEN WITH ISHIMOTO'S MECHANICAL ACCELEROGRAPH.

successive maxima of acceleration) is almost impossible. Thus, a seismograph is practically useless for the determination of these two important items urgently needed by the engineer, whereas they are available with an accelerograph.

Although it is only a short time ago that these accelerographs were installed in our Institute—as after-shocks of the great 1923 earthquake have frequently taken place of late in the marginal region of the disturbed area—we have succeeded in obtaining valuable information on two quite severe earthquakes from them. One was the severe earthquake of June 17, 1931, whose epicenter was about 40 km. north of Tokyo. Its acceleration diagram recorded by an Ishimoto mechanical accelerograph is reproduced in Fig. 35. A record taken by a seismograph which is equipped with an automatic starter and which gives an open diagram, is also shown in Fig. 35. It will be seen that the sharp motion that was the cause of the maximum acceleration was not the conspicuous principal motion of the earthquake, such as *b c d e*, but

most probably the secondary motion, such as *a b*, or that part of the principal motion, such as *b c*, on which a secondary motion was most likely superimposed. Nevertheless, the intensity and the period of acceleration are very difficult to estimate, even from such an open diagram as shown in Fig. 35. Still more difficult will be the estimate from an ordinary condensed seismogram. Indeed, an experienced seismologist estimated from a record obtained by a seismograph for the use of geophysicists, that the maximum acceleration

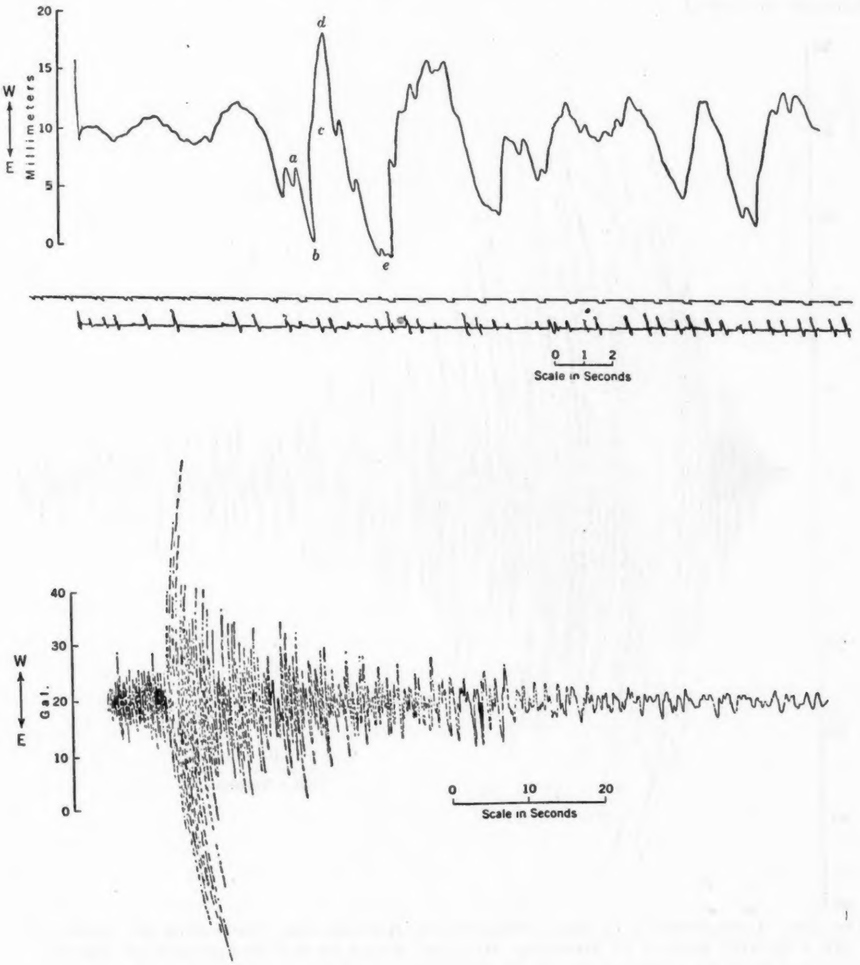


FIG. 35.—ACCELERATION DIAGRAM OF AN EARTHQUAKE ON JUNE 17, 1931.

of that earthquake was 25 gal. and its period, 2.1 sec. As a matter of fact, however, the maximum acceleration of its north and south component of motion was about 43 gal., and its period, 0.4 sec., as recorded by the accelerograph, which had been carefully calibrated both statically and dynamically.

Another earthquake whose accelerogram was successfully taken in our Institute was that which occurred on September 21, 1931, in the Mt. Chichibu region in Northern Musashi, about 60 km. northwest of Tokyo. This earthquake was semi-destructive, and in the alluvial district caused the loss of 16 lives and the collapse of 76 houses, although in a district much nearer the epicentral region, where the ground is of hard paleozoic formation, no casualties were reported. This earthquake shook Tokyo quite severely, although, except for a few cases of cracks in the pavement in the low ground, no serious damage occurred.

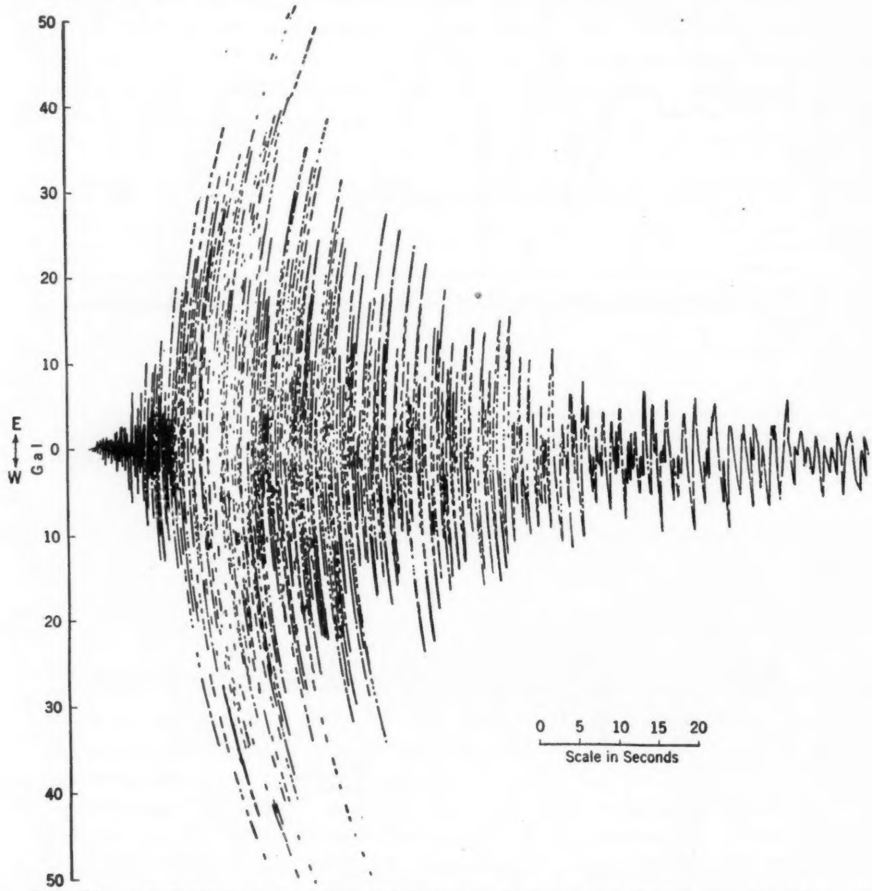


FIG. 36.—ACCELEROGRAM OF SEMI-DESTRUCTIVE EARTHQUAKE, SEPTEMBER 21, 1931, IN MT. CHICHIBU REGION IN NORTHERN MUSASHI ABOUT 60 KM. NORTHWEST OF TOKYO.

The accelerogram of this earthquake is reproduced in Fig. 36, to compare with the seismogram (Fig. 37) taken with an Omori strong-motion recorder. It should be stated that this earthquake occurred on a warm day, with the result that the oil damping of the accelerograph was somewhat insufficient, although not to the extent of rendering the record untrustworthy. It will be seen that the east and west component of acceleration is about 70 gal.

and its period about 0.4 sec. The seismogram, however, gives no useful information except that the ground moved about 3.5 cm. in the principal motion. It is interesting that no casualty occurred in Tokyo notwithstanding the fact that the acceleration attained was so intense that the east-west component was 70 gal. and the other component (the accelerogram of the north-south component is not shown) was 60 gal.

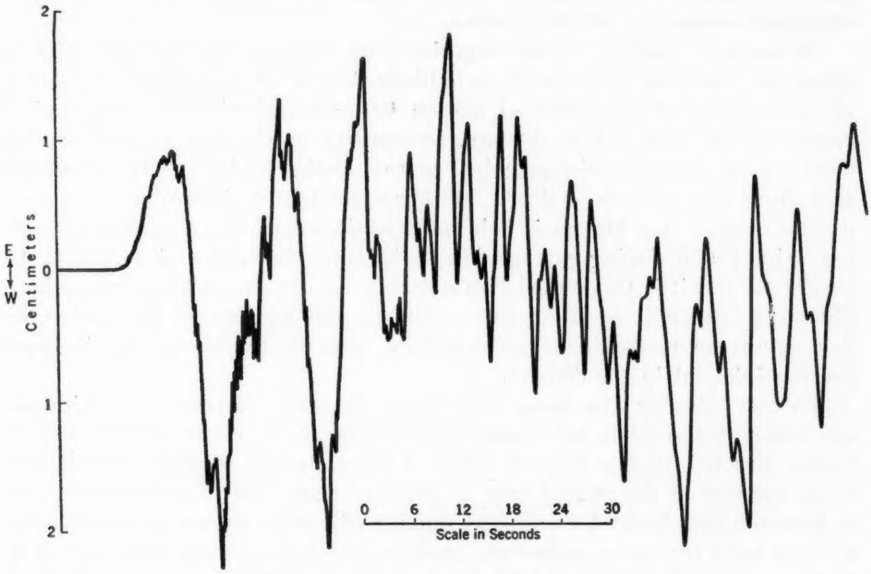


FIG. 37.—SEISMOGRAM OF EARTHQUAKE OF SEPTEMBER 21, 1931, TAKEN WITH AN OMORI STRONG-MOTION RECORDER.

Although, as the recording drum of the accelerograph was driven slowly, the relative phase of the acceleration components could not be detected, so that the intensity of the maximum resultant acceleration is unknown, yet there is no doubt that the intensity of acceleration was between 70 and 92 $(= \sqrt{70^2 + 60^2})$ gal. Nevertheless, this intensity corresponds to strength, IX, of Cancani's seismic scale. Therefore, if the seismologists were right, the earthquake should have caused a catastrophe, but actually it was not so. This fact is worth the special attention of engineering seismologists.

Also, my view that the acceleration of the great 1923 earthquake in up-town Tokyo was more than, say, 0.15 g, as against the general belief that it was 0.1 g, seems to be supported by the observations of this earthquake. Although at the time of this earthquake there was installed in the low grounds a similar accelerograph, it is unfortunate that a record from it could not be obtained on account of the break-down of the recording points by the shock, suggesting the important fact that the intensity was greater down town than up town.

It is to be remarked that not only the three earthquakes whose records are reproduced in Figs. 34, 35, and 36, but all acceleration records so far

taken showed a similar result to the effect that the period of acceleration is not the same as the period of motion, as apparently indicated in a seismogram. This is only natural in view of what has just been mentioned regarding the main and secondary motions of an earthquake. In specifying the intensity of an earthquake, the amplitude and the period of the most conspicuous motion are generally given; but, as a rule, such a motion does not cause the maximum acceleration, and it seldom appears in an accelerogram as a predominant component of acceleration.

Moreover, according to our experience on Tokyo's high ground (Hongo, where our Institute is situated), it is likely that in all earthquakes the period of acceleration of the principal motion was confined within a very limited range of, say, from 0.3 to 0.4 sec., irrespective of the fact that the period of the main waves of the principal motion varied widely. This is clearly seen from Figs. 34, 35, and 36. With respect to this important fact, it is worthy of note that Hongo is habitually subjected to micro-tremors at ordinary times, and during earthquakes to habitual motions with a period 0.3 sec., or so, and it is this period that prominently appears in the accelerations. Therefore, although I cannot say positively, the motions of an earthquake that cause the predominant accelerations seem to be due to the habitual motions inherent in the district.

To test whether the same rule holds in other districts, an Ishimoto accelerograph has lately been installed at Marunouchi, the business center of Tokyo, situated on low ground, where I have already studied the behavior of the motions of the ground both at ordinary times and in earthquakes. As we have not yet obtained a sufficient number of accelerograms in this district, it is too early for me to deduce any positive conclusions from them; but it is almost certain that nothing will develop to contradict the results obtained from investigations on high ground. It is very important to note that on low ground, the period of acceleration of an earthquake of intensity, say, "moderate" and higher, is not uncommonly close to 0.7 sec., while in small earthquakes belonging to the "feeble" class, the period of acceleration is generally 0.2 or 0.4 sec., according to their sharpness. These three different periods correspond closely to the periods of the habitual motion of the ground.

Thus, so far as our latest experience goes, we must radically modify our views on earthquake motions. The amplitude and the period of the main principal motions taken from a seismogram have generally not much significance for the engineer. The requisite data for him are the acceleration and its period as recorded directly by an accelerograph suitable for engineering use.

Here I say emphatically "an accelerograph suitable for engineering use," because our object is achieved only by accelerographs having suitable characteristics. As the acceleration given by harmonic motions is inversely proportional to the square of their periods, it may happen, particularly in the case of earthquakes of near-by and shallow origin, that motions having very short periods (say, less than 0.1 sec.) give the maximum acceleration of an earthquake.

Now, the elementary principle of forced harmonic vibrations tells us that an accelerograph having a damping of, say, 0.7 times the critical value, gives practically correct indication, if the period of excitation almost exceeds two times the undamped natural period of the instrument, and also that it gives reduced indication for quicker motions, the general feature being as shown in Fig. 38. Therefore, if an accelerograph having a very short natural period, such as that sometimes used for geophysical study, is employed, then the record obtained may be only a train of extremely sharp waves, making the detection of more important components practically impossible.

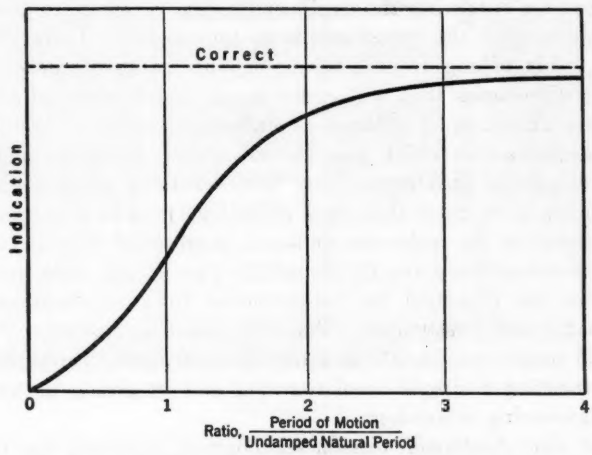


FIG. 38.—RELATION BETWEEN RATIO OF PERIOD OF MOTION TO NATURAL PERIOD OF AN ACCELEROGRAPH, AND THE INDICATION OF THE INSTRUMENT.

Motions having extremely short periods are not of interest to structural engineers, while motions having somewhat longer periods, say, more than 0.3 sec., or so, are of importance. Therefore, for engineering purposes, accelerographs having a suitable period, say, 0.15 sec., or so, must be used; if not, the record of motions in which we are interested will be masked by less important rapid ones, and the record will be nearly as bad as the condensed record of a seismograph, in which, on the contrary, important waves are masked by slow main motions.

Now, reverting to the question under consideration, although the general question of this habitual motion is still *sub judice,* yet the fact that the period of acceleration of earthquakes on Tokyo high ground is generally between 0.3 and 0.4 sec. is very serious to us, because masonry buildings on the high ground are generally of three or four stories in height, and, as will be shown later, their natural periods of vibration are generally of the same order of magnitude as that of the earthquakes. It is also disconcerting to us that in Tokyo the low ground, where comparatively tall buildings (although low for this country) having eight stories or more are standing,

often has an acceleration period of about 0.7 sec., so that the free vibration of such buildings, if not made especially strong, is nearly co-periodic with this period of acceleration of the ground.

I am not well informed as to the nature of earthquake motions in the United States. Presumably, it may not be very different from that in Japan. If so, sky-scrapers that have generally a long natural period of more than 2 sec. seem to be very favorable in this connection. I cannot see the reason why they are sometimes made intentionally more flexible, irrespective of the fact that they are intrinsically flexible.

Nevertheless, I wish to impress on you the fact that the use of the accelerograph is urgent for obtaining the engineering data of an earthquake; for our object it far surpasses the seismograph in importance. I am sure that by pursuing the study of engineering seismology by means of the accelerograph we may make discoveries that will cause us to modify some of our accepted ideas. To give an example, offhand, of an erroneous idea, I take Cancani's well-known seismic scales which give the seismic accelerations and their corresponding damage to buildings. The faith of some seismologists in this scale of intensity is so great that they ridiculously make distorted computations of acceleration in order to make it correspond to the scale. The intensities of acceleration given in the middle part of the scale were probably computed from the principal motion recorded in a seismogram, and are, therefore, greatly under-estimated. The scale must be revised. With regard to the problem under consideration, I highly appreciate Professor Ishimoto's work in constructing a simple accelerograph, and in giving us useful information on engineering seismology.

The Wood and Anderson torsion-seismograph invented in the United States seems to be one of the best instruments for use as an accelerograph, and I highly recommend the distribution of the instrument in every seismic district for installation, both directly on the ground and in the principal buildings.

(V) The Period of the "Natural" Ground Motion

It is hardly necessary to say that the period of an earthquake motion, or in a stricter sense, the period of accelerations, plays an important rôle in the destructive effect of an earthquake on buildings and other structures. It is important, therefore, to investigate this period of habitual motion peculiar to the ground, if such motion does really exist, at ordinary times and during earthquakes. At first thought, it seems rather odd to speak of a period of the ground when the ground has practically unlimited extension both laterally and downward; but when we remember that the ground of a district like Tokyo, which is of sedimentary formation, is made up of several strata, it is not difficult to understand the existence of a period of motion peculiar to the district. In a mathematical paper entitled "Possibility of Free Oscillation of the Surface Layer Excited by Seismic Waves,"[9] Professor K. Sezawa, of our Institute, showed the possibility of motions having a natural period proper to the surface layer of a stratified crust, and also that the period

[9] *Bulletin,* Earthquake Research Inst., Vol. 8 (1931), 1.

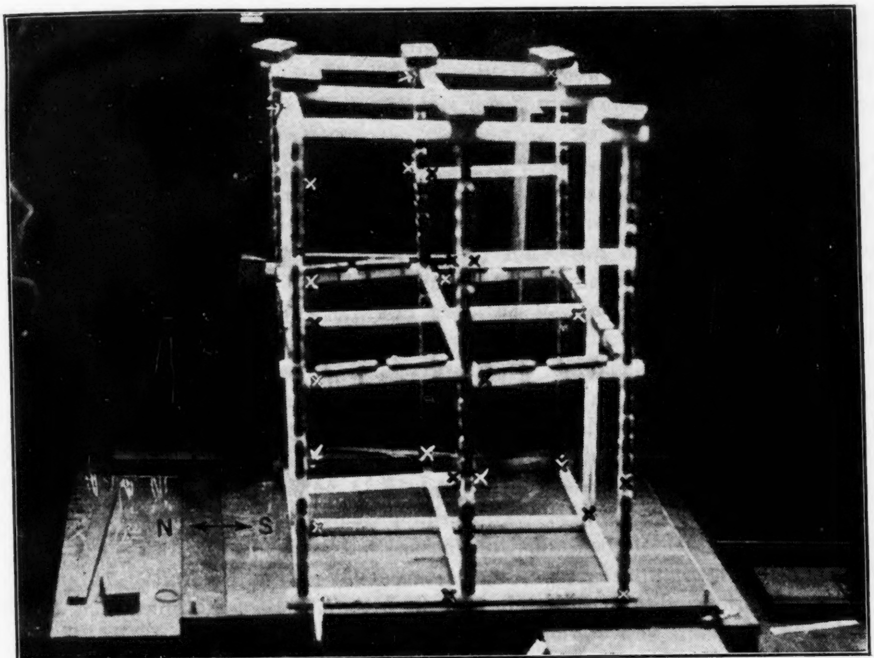


FIG. 39.—MECHANICAL MODEL OF A WOODEN FRAME BUILDING (x INDICATES POINT OF BREAKDOWN).

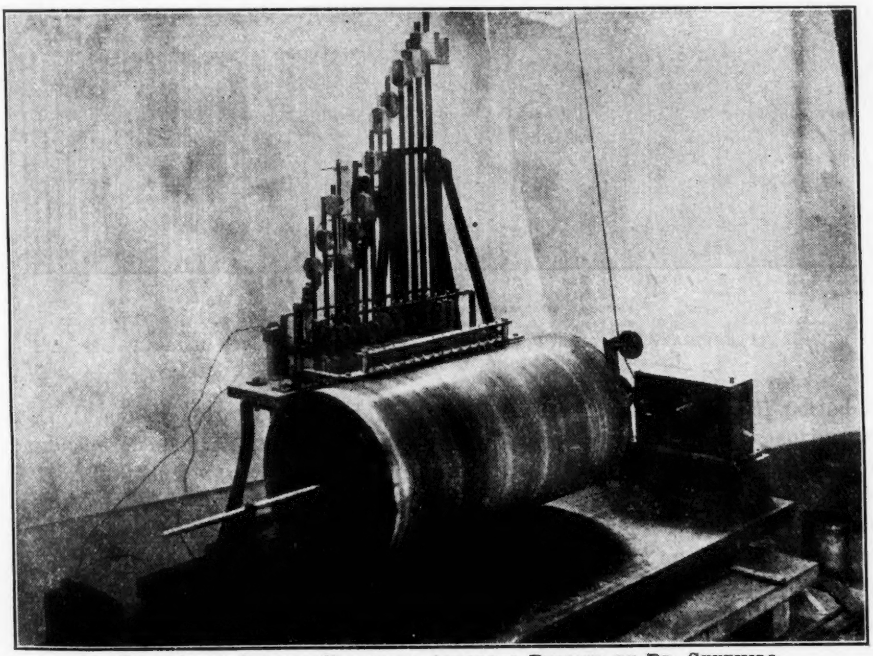


FIG. 40.—A SEISMIC VIBRATION ANALYZER DEVISED BY DR. SUYEHIRO.

depends on the nature of the crust forming the strata as well as on the thickness of the surface layer. Not only has the mathematical solution proved the possibility, but observations have actually shown the existence of these habitual ground motions. (Fig. 39 is a mechanical model of a wooden frame building, discussed later.)

For detecting the prevalent period, if any, of earthquakes in a particular locality, I have used a seismic vibration analyzer, devised by myself.[10] This instrument works on the principle of selective resonance as in Hartmann's well-known reed frequency-meter. As shown in Fig. 40, the analyzer consists of a number of compound pendulums having different natural periods, the shortest period being 0.2 sec. and the longest 1.8 sec., and they are arranged side by side in a row along the side of a recording drum. Each pendulum is provided with a separate water damper to wipe out the free oscillation, and each damper is so adjusted as to make the magnification of the amplitude of the resonance vibration of each stylus practically equal. The instrument shown in Fig. 40, although very clumsy in appearance, works satisfactorily. The making of a new one of smaller size and more finished appearance is now under contemplation.

A typical record with this instrument obtained at Hongo is shown in Fig. 41. In this diagram the numerals at the top show the natural periods of the pendulums. The record clearly shows that only motion having a period of 0.3 sec. persisted, and that this motion was likely to be of the harmonic type. The other motions gave merely inconspicuous and irregular, jagged undulations. This feature is characteristic of all earthquakes in Hongo, irrespective of their intensity and the distance from the origin.

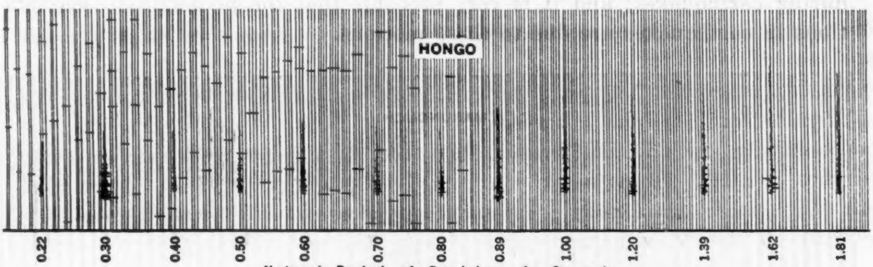


FIG. 41.—TYPICAL RECORD OBTAINED BY SUYEHIRO VIBRATION ANALYZER AT HONGO, ON HIGH GROUND OF TOKYO.

It can be inferred, therefore, that this district has a natural period of motion of about 0.3 sec. When Professor Ishimoto and I were measuring the vibration of low monolithic buildings with a mercury tube microvibrograph designed by us, we took records at the same time of micro-tremors of the ground on which the buildings stood. In one of them a record of the micro-tremor of the northern part of Hongo was obtained. By analyzing those parts of the record that were undisturbed by passing traffic, I obtained curves

[10] For details, see, "A Seismic Vibration Analyser and the Records Obtained Therewith," *Bulletin*, Earthquake Research Inst., Vol. 1 (1926).

showing the frequency of occurrence of motions having different periods, one of which is shown in Fig. 42. The diagram shows that the motion having a period of 0.3 sec. has the maximum frequency of occurrence. A similar observation was made later by Professor Ishimoto and Professor Takahashi with an Ishimoto microvibrograph on the grounds of our Institute, which is

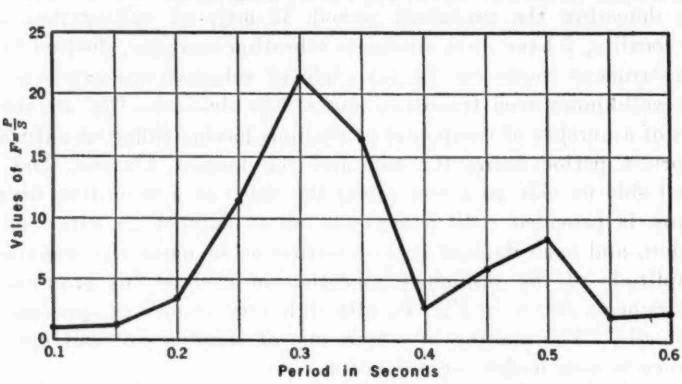


FIG. 42.—DISTRIBUTIONS OF VARIOUS PERIODS OF TREMORS (F = FREQUENCY OF OCCURRENCE; P = PERIOD; AND S = STANDARD PERIOD OF 0.3 SEC.).

situated 2 km. south of the place mentioned, when exactly the same behavior of the ground was observed.

Thus, it will be seen that Hongo, on the high ground of Tokyo, has a habitual motion with a period of about 0.3 sec., both at ordinary times and during earthquakes; and it is very probable that the predominant accelerations in earthquakes are due to these motions.

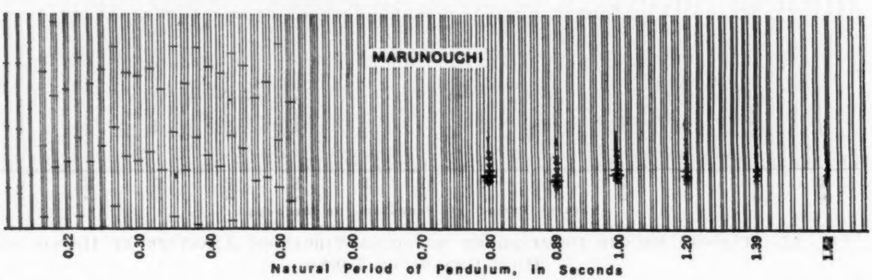


FIG. 43.—TYPICAL VIBRATION ANALYZER RECORD TAKEN AT MARUNOUCHI IN THE LOW PART OF TOKYO.

I have carried out similar observations at Marunouchi, on the low ground of Tokyo. There, matters are not so simple as at Hongo. The seismic vibration analyzer indicates that the prevailing periods in earthquakes are generally from 0.7 to 0.9 sec. (in the record shown in Fig. 43, 0.8 sec. predominates). This accords well with the fact that the period of acceleration of moderate earthquakes in that district is sometimes about 0.7 sec., or more; but as shown in Fig. 44 (which is a reproduction of a record of the micro-

tremor taken with an Ishimoto microvibrograph by Professor Takahashi, Mr. K. Sato, and myself), the prevailing period of the micro-tremor in that district is from 0.4 to 0.45 sec., being nearly one-half the prevailing period in an earthquake. With these are mixed some motions having smaller amplitudes, and periods varying from 0.2 to 0.3 sec. Remembering that the period of acceleration of feeble earthquakes in Tokyo low ground is sometimes 0.2 to 0.4 sec., it can be inferred that, according to circumstances, the secondary and tertiary free motions of this district are excited by minor but sharp earthquakes.

Thus, this district is remarkable in that there exist secondary and tertiary motions, having natural frequencies of twice and three times the motion that appears to be the fundamental one. Such behavior of the ground is not conceivable from the mathematical solution for an elastic ground, just cited,

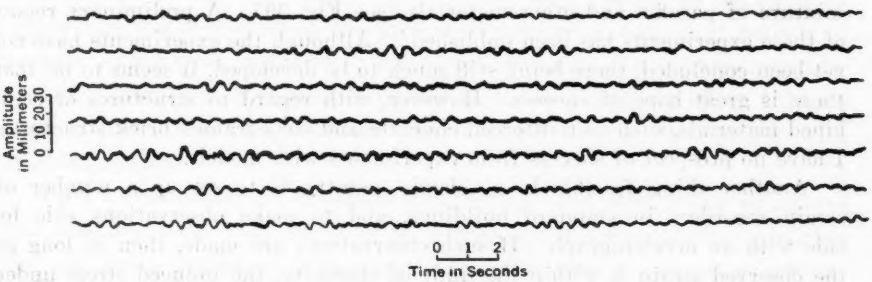


FIG. 44.—RECORD OF MICRO-TREMOR TAKEN IN THE LOW PART OF TOKYO WITH AN ISHIMOTO MICROVIBROGRAPH.

as given by Professor Sezawa. As the district was once marshy, and as the earth is oozy even now, it is not unreasonable to consider the ground as having the character of a semi-fluid. If so, a district in Tokyo the surface and subsurface stratifications of which are very irregular, may have a multiple periodicity in harmonic order just like water in a tank. In my estimation, the fundamental motion has a period of about 0.8 sec., but the period is too long to be excited by traffic and other minor disturbances, and it is only in the case of earth movements having a period approaching 0.8 sec. that the fundamental motions come out clearly. However, I shall not go further into this question, but shall leave it to competent investigators.

The only thing that I wish to emphasize is that every locality has its own "natural" motion during an earthquake, and we must pay careful attention to it.

(VI) STRAIN MEASUREMENT OF BUILDINGS IN EARTHQUAKES

With regard to the strength calculation of buildings and other structures in an earthquake, I had better say nothing; because this country is the birthplace of most of the methods of calculation, such as the "slope-deflection method" by Professor W. M. Wilson, the "portal method," etc. If I do so, I shall just be "carrying coals to Newcastle" as they say in England; but whatever elaborate method of calculation we may use, it is a matter of

utmost difficulty to calculate the accurate stress and strain induced in a member of a complicated statically indeterminate structure under a given seismic force, especially when such a structure is fitted with diagonal bracings, partition walls, and the like. Besides, as an actual problem, we have to consider other important items, such as the relation between the natural period of the structure and the period of acceleration, the damping against vibrations, mutual action between the foundation of the structure and the ground bed, etc., some of which will be taken up as subjects of Lecture III. If we take all these matters into consideration, the strength calculation is virtually impossible.

One feasible step in this direction is to make experiments with models satisfying the law of mechanical similitude. I am now making experiments for simple wooden frame structures with a mechanical model made of a mixture of paraffin and other materials (see Fig. 39). A preliminary report of these experiments has been published.[11] Although the experiments have not yet been concluded, there being still much to be developed, it seems to me that there is great hope of success. However, with regard to structures of combined materials, such as reinforced concrete and steel-framed brick structures, I have no prospect of success from experiments with models.

Another thing feasible in a seismic country is to set up a number of strain recorders in standard buildings, and to make observations side by side with an accelerograph. If such observations are made, then as long as the observed strain is within the limit of elasticity, the induced stress under a known acceleration can be estimated. Fortunately, in any district, the period of acceleration is practically fixed. Therefore, we can estimate under what earthquake intensity the construction members exceed the elastic limit (for steel structures), or crack (for masonry structures), although for obvious reasons we cannot estimate the break-down of steel structures; for this, calculations are equally impossible.

I regret, however, that having been kept very busy of late on account of the occurrence of severe earthquakes one after another, as was mentioned in Lecture I, I could not fully extend my researches to this problem. At present, I have only one strain-recorder fitted to a low wooden building; the data for such a building are evidently of no use to you; but I am now ready to extend the strain measurements to high buildings, and hope that at no distant date I shall be able to supply useful data covering such buildings.

[11] *Proceedings*, Imperial Academy, Vol. 6 (1930), No. 7.

LECTURE III
VIBRATIONS OF BUILDINGS IN AN EARTHQUAKE

Synopsis

The behavior of vibrating buildings and the mutual action between their foundations and the adjacent ground, are the subjects of Lecture III. Dr. Suyehiro gives the results of actual observations on buildings of different types of construction and emphasizes the features of certain buildings that explain why they withstood the great 1923 earthquake. He also points out why certain buildings failed.

A rigid building was found to move in the same manner as the adjacent ground and was insensible to the ground vibrations of very short periods of the order of 0.1 sec. In the less rigid buildings the motion is complicated by secondary vibrations, and such buildings have less damaging effect. Consequently, their own free vibration predominated during earthquakes with irregular motions.

Wood frame buildings—carefully constructed—and steel frame structures with masonry walls were found to have high earthquake-resisting qualities. The cushioning action of the ground may serve more or less to relieve the destructive action of a strong earthquake in the case of masonry buildings in which the weight is relatively great. In Tokyo, buildings with monolithic, flat-slab foundations and without piles, but located on soft, compacted soil, withstood shocks better than buildings with individual pile footings. Studies of underground earthquake motions led to the opinion that the idea of extending the footings of a tall building, with a long natural period of vibration, deep into the ground is not so advantageous as is believed by some engineers. For low buildings with a short natural period, on the other hand, the advantages of deep footings are unquestioned.

Introduction

The structural members of a building in a seismic country must have sufficient strength to withstand not only the static vertical forces due to dead and live loads, but also the horizontal dynamic forces due to earthquake motion. In most seismic countries, therefore, the seismic force to be considered in the strength calculation is stipulated in the building code. For instance, in Japan, it is required that the horizontal seismic acceleration must be taken as at least one-tenth of the acceleration due to gravity. The horizontal force due to this acceleration is included in the strength calculation as if it were a sort of wind pressure, but with the slight difference that in this case the horizontal force is proportional to the weight of the structural

members and acts on every part of the building. Thus, it would seem that determination of the scantlings of the structural members of an earthquake-resisting building is achieved in a simple manner.

On reflection, however, we find that the problem is not so simple as it appears at first sight. First, the prescribed intensity of the maximum seismic acceleration which forms the basis of the strength calculation is nothing but an arbitrarily assumed quantity. This seismic intensity was not fixed either by past experience or because of possible occurrence in the future. Perhaps it is because one-tenth is a convenient number, seeing that the sizes of members calculated from it generally turn out to be moderate. There is no sound scientific basis for adopting this seismic factor, except that as mentioned in Lecture II, it is known in a hazy way that, at the beginning of the principal motion of the 1923 Kwanto earthquake, an acceleration of this order is likely to have taken place in the higher ground of Tokyo; but there are reasons to believe that the actual intensity was somewhat greater. Until we succeed in determining the real intensity of a destructive earthquake in a scientific manner, we are not in the position to say positively whether or not this seismic factor is the proper one to use, except from a practical point of view. Thus, the most urgent necessity is to be prepared for measuring the intensity when the next severe earthquake comes.

No less important is the study of the vibration of buildings in an earthquake, whether severe or otherwise. Here, the behavior of the vibrating buildings and the mutual action between the foundation of the building and the ground are the items to be investigated. Evidently, these two questions are intimately correlated and need to be considered together.

(I) Free Vibration Periods, the Effect of the 1923 Earthquake, and Other Observed Data Concerning the Behavior of Buildings

For the problems now under consideration, it is necessary, first, to know the period of the free vibration of actual buildings, because their vibration (free or forced), in an earthquake largely depends on the ratio of the natural period of the buildings to that of the imposed vibration, and also upon the damping of the vibration, which is closely related to the natural period.

In Japan, buildings taller than 100 shaku (1 shaku = 0.994 ft.), irrespective of type of construction, are prohibited. Consequently, our data may possibly give little information useful to American engineers, whose principal interest is to build sky-scrapers; yet it may not be out of place to present some of these data, as my discussion refers to them from time to time.

In order to give an idea of the effect of the 1923 earthquake on the buildings listed in Table 3, I will show photographs of some of them taken after the earthquake, and will make a brief explanation of them. Fig. 45 shows one of the burned districts in Tokyo. In the center stands the Ryounkaku Tower (No. 15, Table 3) ruined by the earthquake. It will be noted that, except for a brick building on the right, no houses are to be seen in the background, showing how the fires that followed the earthquake converted this quarter of the city into a barren field. In contrast, Fig. 46 shows the

TABLE 3.—USEFUL INFORMATION CONCERNING SOME IMPORTANT BUILDINGS IN JAPAN

No.	Name of building	Construction	Sizes: Height above street level, in feet	Sizes: Floor area, in square feet	Stories	Ground	Condition	When measured	Periods, in Seconds: Longitudinal	Periods, in Seconds: Transverse	Observer	Type of instrument	Remarks
							STEEL FRAME BUILDINGS						
1 (a)		Office building.					Nearly completed.	Before the semi-destructive earthquake of April 26, 1922.	0.94	0.89	Omori	Seismograph	
1 (b)		Brick and hollow-tile					Slightly damaged.	Just after this earthquake.	1.01	1.09	Omori	Seismograph	Suffered moderate damage by the great earthquake
1 (c)	Marunouchi Building	Curtain wall.	109	67 600	Nine and cellar	Alluvial	Repaired and strengthened.	When repairs nearly completed.	0.51 0.71	0.67	Omori	Seismograph	
1 (d)		Separate foundation.					Moderately damaged skeleton intact.	Just after the 1923 earthquake.	1.18 0.48	1.11 0.50	Horikoshi	Seismograph	
1 (e)		Reinforced concrete wall.					Thoroughly repaired and additionally strengthened.	When the second repairs completed.	0.25	0.23	Saita	Seismograph	
2 (a)	Tokyo	Restaurant: Brick curtain wall.	86	18 200	Five and cellar	Alluvial	Nearly completed.	Before the great earthquake.	0.72	0.54	Omori	Seismograph	Severely damaged
2 (b)	Kaikan	Separate foundation.					Severely damaged.	Just after the great earthquake.	1.30	1.20	Horikoshi	Seismograph	
3 (a)		Office building: Brick and hollow-tile curtain wall.					Completed.	Before great earthquake.	0.69	0.77	Omori	Seismograph	Fairly serious damage
3 (b)	Yusen Building	Separate foundation.	100	43 400	Seven and cellar	Alluvial	Severely damaged.	After great earthquake. During repairs.	0.90	0.80	Saita	Seismograph	
4 (a)		Office building: Brick and hollow-tile curtain wall.					Completed.	Before the great earthquake.	.....	0.61	Omori	Seismograph	
4 (b)	Yurakukan Building	Separate foundation.	100	22 600	Seven and cellar	Alluvial	Moderately damaged.	Just after great earthquake.	0.80	0.80	Taniguchi	Seismograph	Moderate damage
4 (c)		Partly reinforced concrete wall.					Additionally strengthened.	Repairs completed.	0.55	0.45	Taniguchi	Seismograph	

TABLE 3.—(*Continued*)

No.	Name of building	Construction	Size		Stories	Ground	Condition	When measured	Periods, in Seconds		Observer	Type of instrument	Remarks
			Height above street level, in feet	Floor area, in square feet					Longitudinal	Transverse			
							STEEL FRAME BUILDINGS (*Continued*)						
5	Kaijo Building	Office building: Brick and reinforced concrete curtain wall. Foundation connected.	87.5	26 500	Seven	Alluvial	Long completed.	Before the great earthquake.	.....	0.45	Omori	Seismograph	Slight damage
6	Kogyo Bank Building	Office building: Brick and reinforced concrete curtain wall. Foundation connected.	98.5	23 500	Seven and cellar	Alluvial	Long completed.	Before the great earthquake.	0.61	0.65	Omori	Seismograph	Very slight damage
7	Ginza Building	Department store. Reinforced concrete wall.	100	22 000	Eight	Alluvial	....	After completion (After the great earthquake)	0.70	0.70	Saita	Seismograph	Half finished. No damage
8	Marunouchi Hotel	Reinforced concrete wall.	100	6 580	Nine	Alluvial	....	After completion (After the great earthquake)	0.60	0.50	Saita	Seismograph	Half finished. No damage
9	Earthquake Research Institute	Reinforced concrete wall.	31	5 170	Two and two cellars	Diluvial	....	After completion.	0.30 (not elastic vibrations)	0.30	Ishimoto	Micro-vibrograph	Built after the great earthquake
							REINFORCED CONCRETE BUILDINGS						
10	Naigai Building	Office building: Reinforced concrete frame with brick filler wall. Separate foundation	100	21 950	Seven	Alluvial	Nearly completed	Before the great earthquake.	0.65	0.65	Nagata	Seismograph	Collapsed
11	Nippon Bank Annex	Office building: Reinforced concrete frame with brick filler wall	100	11 300	Seven	Alluvial	Long completed	Before the great earthquake.	0.48	0.43	Nagata	Seismograph	Moderate damage
12	Tokyo Nichi-nichi Shinbun Sha	Newspaper office: Reinforced concrete frame and wall	80	{ 2 860 2 860 5 190	Two Four Five	Alluvial	Long completed	Before the great earthquake.	0.52	0.52	Nagata	Seismograph	Slight damage
13	Old Aeronautical Laboratory	Reinforced concrete frame and wall	36	7 500	Two	Reclaimed	Long completed	After the great earthquake.	Transversal Fundamental, 0.50	Transversal Secondary, 0.10	Ishimoto and Suyehiro	Micro-vibrograph	No damage

TABLE 3.—(*Continued*)

No.	Name of building	Construction	Size		Stories	Ground	Condition	When measured	Periods, in Seconds		Observer	Type of instrument	Remarks
			Height above street level, in feet	Floor area, in square feet					Longitudinal	Transverse			
							Reinforced Concrete Buildings (*Continued*)						
14	Mitsubishi Laboratory	Reinforced concrete frame and wall	33	7 500	Two and cellar	Diluvial	Long completed	After the great earthquake.	Transversal Fundamental, 0.35	Transversal Secondary, 0.15	Ishimoto and Suyehiro	Microvibrograph	No damage
							Brick Buildings						
15	Ryounkaku Tower	Observation tower, octagonal section. Small wooden two-storied superstructure on top	130 (main structure)	40 (in external diameter)	1.0 (main structure)	Alluvial	Long completed	August 5, 1919	1.08		Omori	Seismograph	Collapsed
16	Natural History Museum, Tokyo University	Wooden floor; high gables	37	4 700	Two	Diluvial	Long completed	1902	0.33	.....	Omori	Seismograph	Severe damage
							Wooden Buildings						
17	Asakusa Pagoda, Tokyo	Temple tower; four strong columns on each side	81.7	16 (basement story) 10 (fifth story)	Five	Alluvial	Built in 1692	1919	1.35		Omori	Seismograph	Absolutely intact in all earthquakes
18	Japanese hotel in Idu	Japanese style; frames are strongly connected	40	4 450	Three	Diluvial	Nearly completed	After the 1930 Idu earthquake.	0.7	0.07	Saita	Seismograph	Intact in the Idu earthquake
19	Japanese dwelling house in Idu	Ordinary Japanese style	18	930	Two	Alluvial	Long completed	Before the 1930 Idu earthquake.	0.48	0.52	Suyehiro	Seismograph	Intact in the Idu earthquake
20	Naval Architectural Laboratory Annex	Western style	15.5	4 440	One	Diluvial	Long completed	1930	0.3	0.5	Suyehiro	Vibration analyzer	Intact in the great 1923 earthquake

Marunouchi Quarter, the most unharmed part of the city. It is the business center of Tokyo. Except for the unsightly appearance of the Naigai Building (No. 10, Table 3), no one would suppose, without careful examination, that these views were taken just after the great earthquake and the fires that laid waste the greater part of the city. This remarkable contrast is the natural consequence of the fact that in the former district most of the houses were of fragile wooden or brick construction having little resistance against earthquake and fire, while in the latter district, buildings with a few exceptions (such as the Naigai Building), were built substantially, being owned by the Mitsubishi Company which has the reputation for being one of our soundest business concerns, and which constructs its buildings with strength and with the utmost care.

It is remarkable that, except for the single case of the Marunouchi Building, which was built by a foreign contractor with slight experience in destructive earthquakes, all buildings owned by the Mitsubishi Company were perfectly intact. This building (No. 1, Table 3) is shown in Figs. 46 and 47. About 80% of the brick buildings either collapsed or suffered damage in the great earthquake. One of these was the office of the Yeiseikai (the Hygienic Society). This house was built in the German fashion, with thin walls, and had not been provided with sufficient margin of strength against earthquakes.

It is interesting to quote a passage regarding the old Engineering Building of the Tokyo Imperial University, from Milne's "Seismology." He says:

> "Certain roofs which are of considerable span in the old Engineering School, Tokyo, were built so that they rested freely upon the supporting walls, and were not carried with them in horizontal displacements. Although during the last thirty years they have experienced many severe shakings, hitherto they have remained uninjured."

Although the merit of such construction is open to question, there is no doubt that some attention was given to the possible seismic effect on the building. Nothwithstanding this fact, the building suffered severe damage as shown in the photograph.

Amidst the failure of the majority of brick buildings, none of those owned by the Mitsubishi Company (Dr. T. Sone, Chief Architect)—one of which is seen in the foreground of Fig. 46—suffered even a single crack, and they stand as monuments of good work. The reason for this is that not only was the construction excellent, but the workmanship was also excellent; the bricks were reinforced by horizontal iron bonding and vertical rods embedded in the joints. The fact that whenever the outdoor temperature fell below a certain point, the brick-laying was stopped, is sufficient to show with what care the construction was executed.

The Yusen Building (No. 3, Table 3) is seen on the left side in the background of Fig. 46, and is shown in Fig. 48. This building was constructed of steel frames with curtain-walls of hollow tiles one and one-half bricks thick, covered with ornamental terra cotta facing. It suffered moderate damage, such as cracks in the external and partition walls, although the steel framing was unharmed. Yurakukan (No. 4, Table 3), Marunouchi

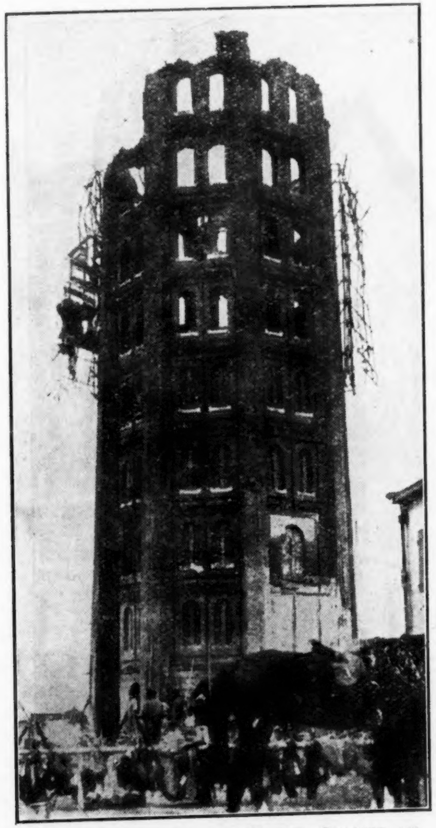


FIG. 45.—RYOUNHAKA TOWER RUINED IN 1923 EARTHQUAKE

FIG. 46.—BUILDINGS IN MARUNOUCHI QUARTER, THE LEAST DAMAGED PART OF TOKYO, TAKEN AFTER 1923 EARTHQUAKE.

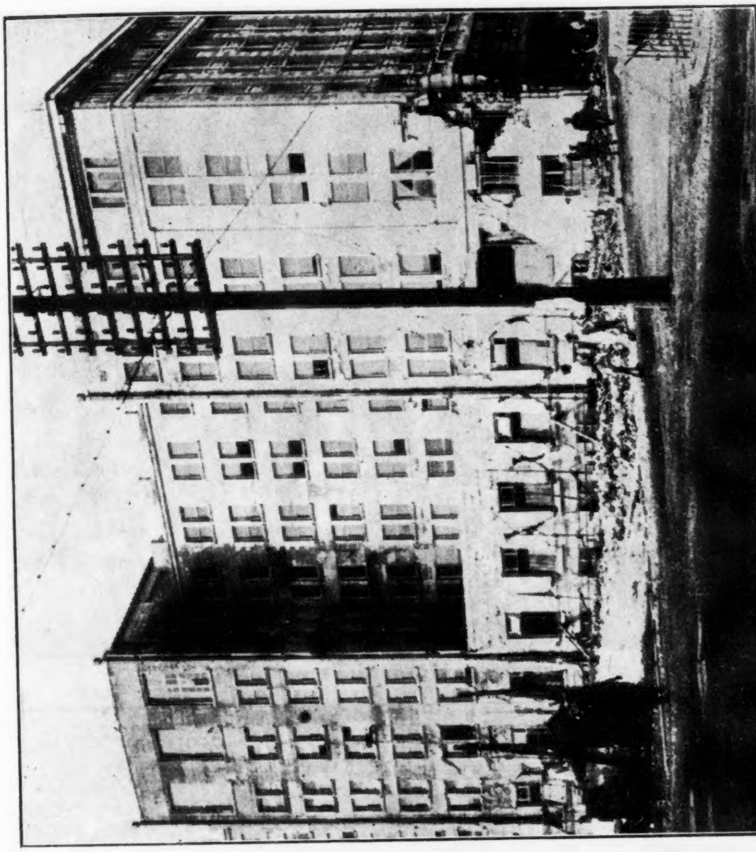


FIG. 48.—THE YUSEN BUILDING AFTER THE EARTHQUAKE OF 1923.

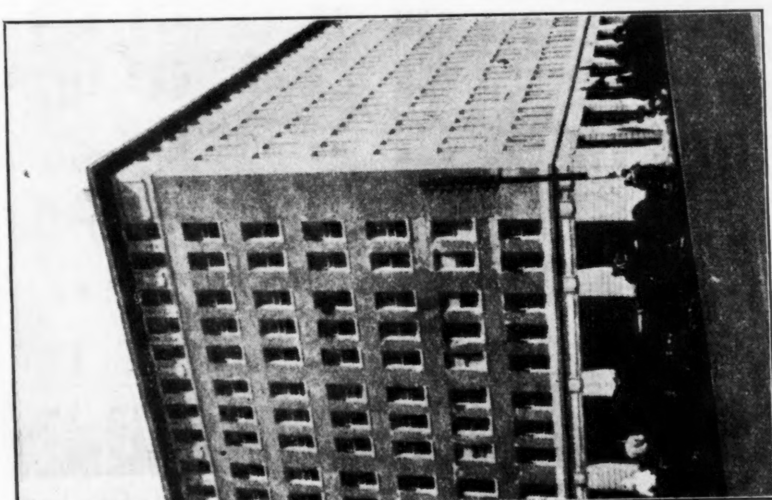


FIG. 47.—MARUNOUCHI BUILDING AFTER 1923 EARTHQUAKE.

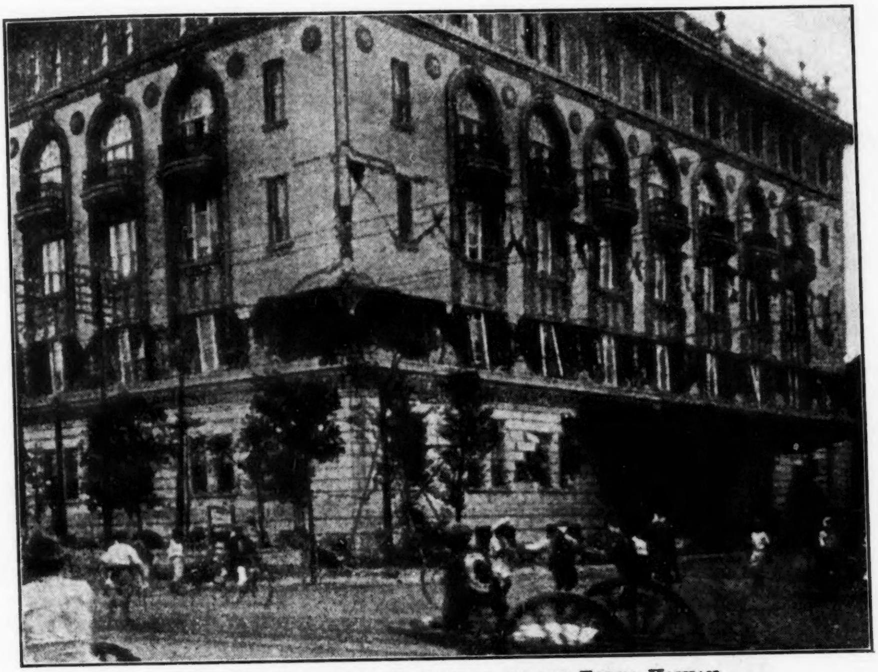


FIG. 49.—VIEW SHOWING DAMAGE TO THE TOKYO KAIKAN

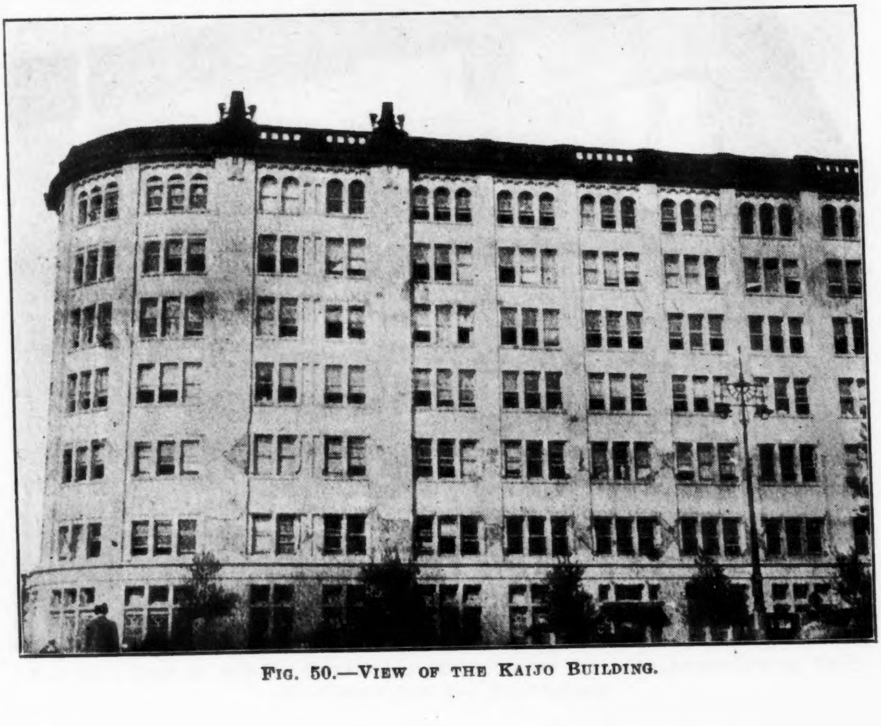


FIG. 50.—VIEW OF THE KAIJO BUILDING.

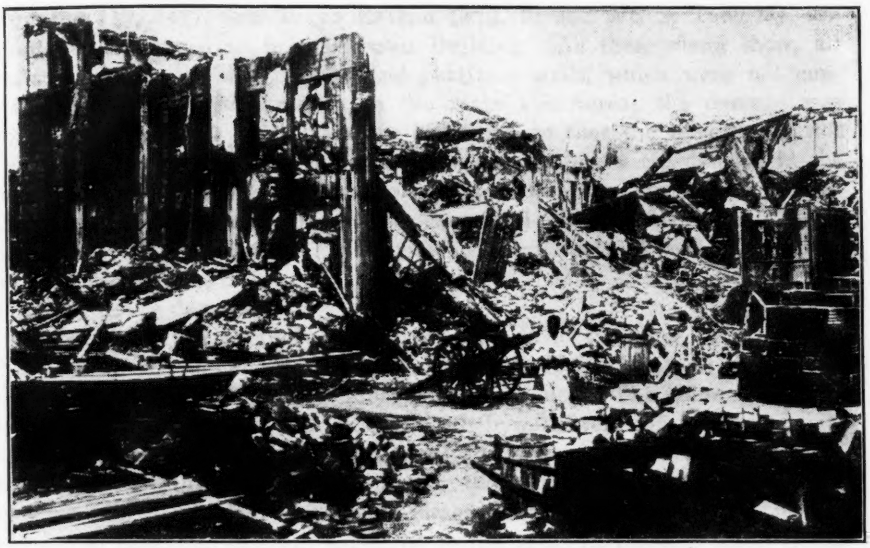


FIG. 51.—RUINS OF THE NAIGAI BUILDING

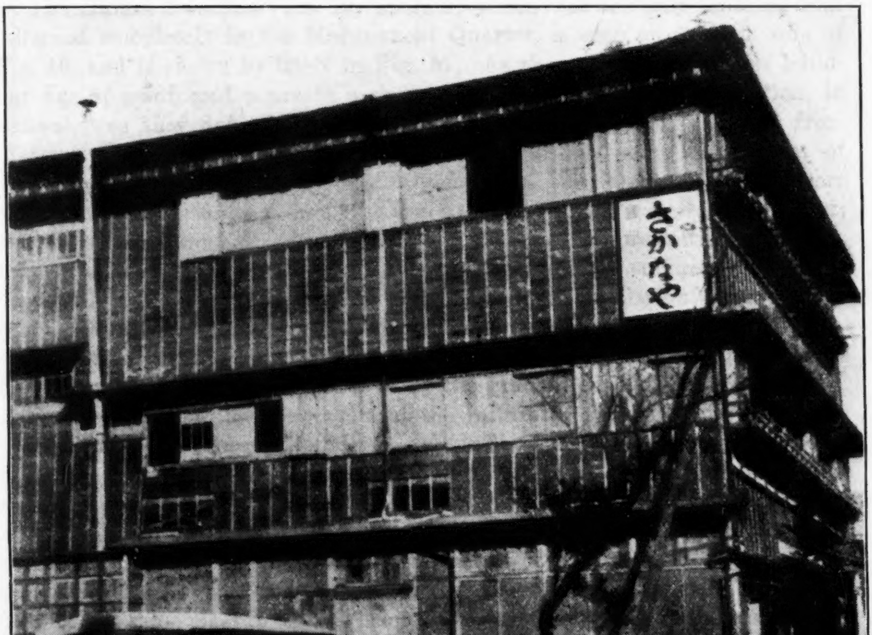
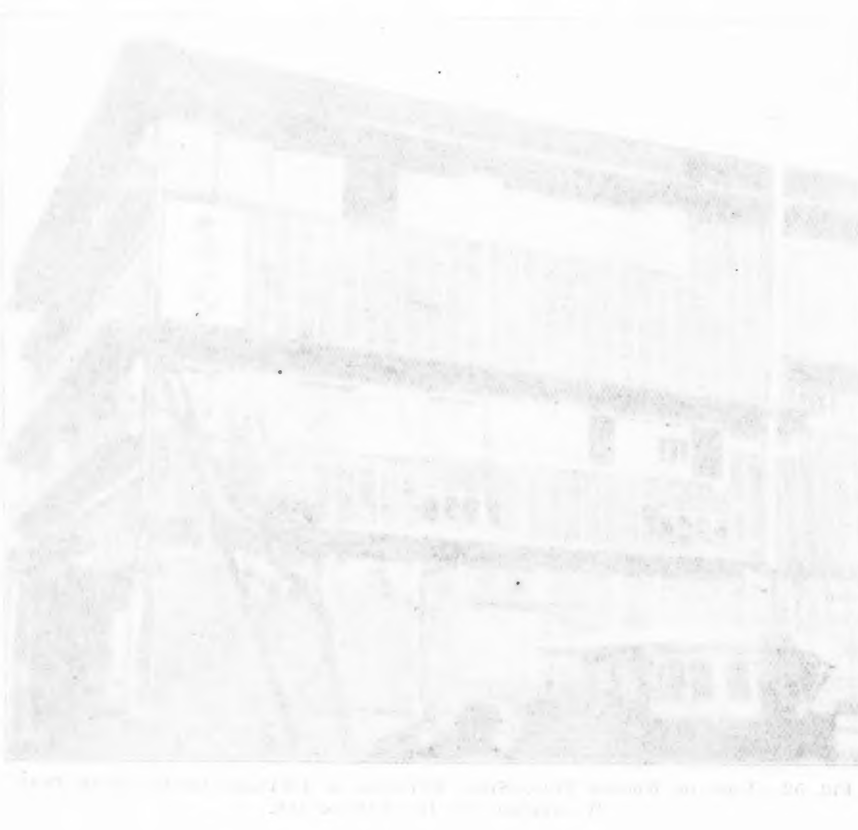


FIG. 52.—VIEW OF WOODEN THREE-STORY BUILDING OF JAPANESE ARCHITECTURE THAT WITHSTOOD THE IDU EARTHQUAKE.



Building (Fig. 47), and Tokyo Kaikan (Fig. 49 and No. 2, Table 3), are similar in construction to the Yusen Building. As these views show, all suffered damage in the external and partition walls, which were not constructed to contribute strength to the main structures; the damage was comparatively slight in the first two buildings, so that the tenants did not have to vacate them during the subsequent repairs; while the damage in the last-named building was serious, the second-story walls being shattered and the columns badly bent. Among these three buildings, the last two had comparatively light framing, but the Marunouchi Building had been additionally strengthened before the great earthquake and had a narrow escape from suffering heavy damage.

Other steel-frame buildings, such as the Kaijo Building (Fig. 50 and No. 5, Table 3), and the Kogyo Bank Building (No. 6, Table 3), were more substantially constructed than those just mentioned. The column footings were well connected with each other, and their walls were constructed of reinforced concrete and bricks, in such a way as to act as more than mere curtain-walls. Naturally, they withstood the earthquake splendidly. The Marunouchi Hotel (No. 8, Table 3), was of steel-framed construction with reinforced concrete, and was subjected to the great earthquake while it was being built. It suffered no damage.

The Naigai Building (No. 10, Table 3) which was the only building that collapsed completely in the Marunouchi Quarter, is seen on the left side of Fig. 46, and is shown by itself in Fig. 51. As shown in Table 3, this building was of reinforced concrete with brick filler walls. The construction, in general, was very defective, the structural design being boldly copied from that in vogue in an aseismic country. The footings were independent of each other; no substantial partition walls were provided; the beams were loosely connected to the pillars; reinforcement bars were not properly joined; and the stirrups were deficient in number. In short, no precaution was taken to provide against a severe earthquake. The building suffered the fate it deserved. It is worthy of notice that the collapse of the building did not occur with the first shock of the earthquake, but nearly 10 sec. after the principal motion had begun. The destruction was probably caused either by the cumulative effect of the vibration or by the gradual development of cracks.

I must tell you, by the way, that the failure of this building and a few other reinforced concrete buildings (all badly constructed) are exceptional cases (in the city, only 7 cases of total failure out of 593 buildings were reported). Therefore, it is a hasty conclusion to judge from these exceptional cases that the reinforced concrete building has little resistance against a severe earthquake. On the contrary, the result of tests with this severe earthquake was fairly good; nearly 78% of this class of buildings in the city, and 75% of the same class in the environs, were perfectly intact in the earthquake. Except those already mentioned, most of the buildings seen in Fig. 46 are of reinforced concrete, and nearly all of them were uninjured.

A wooden building, (No. 18, Table 3) is shown in Figs. 52 and 53. It is a three-story hotel built in the Japanese style. As will be seen from the photograph, it stood unharmed in the general wreckage in the Idu earthquake. This

building not only had strong pillars, extending its entire height, but in the joints of all structural members iron strips and bolts were freely used, mortise, tenon, and joints being avoided as far as practicable. It seems that regardless of the building material, those buildings that are constructed honestly and with good sense can withstand severe earthquakes.

Thus far, I have stated roughly the effects of the great earthquake on some of the buildings listed in Table 3. More complete descriptions of the damages suffered by various kinds of buildings would no doubt be found instructive. However, I must leave such descriptions to the various reports on the 1923 earthquake, which were abundantly published in my country, and among which the Reports of the Imperial Earthquake Investigation Committee, No. 100, C and D, and Reports on the 1923 Earthquake Damages, published by the Civil Engineering Society, Japan, are most commendable. Although these reports are in Japanese, a large part of the volumes consists of photographs and drawings, which are, in effect, an international language, so that there should not be much difficulty in studying them.

((II) Vibration of a Rigid Building in an Earthquake

We shall now proceed to investigate the problem under consideration. To study, in the first place, to what extent the motion of a rigid building follows that of the ground, and, at the same time, to study the nature of its vibration in an earthquake, I have measured the vibration of our Institute building in an earthquake. (See Fig. 54). It is a steel-framed, reinforced concrete structure and is extra strong (weight 53 lb. of steel framing and bars per square foot of floor area). The nature of the vibration of this building in ordinary times was exhaustively investigated[12] by one of my colleagues, Professor Ishimoto. Fig. 55 shows, side by side, the vibrations of the basement floor and the neighboring ground as recorded simultaneously by Ishimoto's microvibrographs with a high magnification (about 3 000 times). It will be seen that the building moved persistently to and fro with a period of about 0.3 sec. Simultaneous observations of the motion in different parts of the building showed that the oscillation was not an elastic one, but very probably the motion of a rigid body on the ground-bed.

For comparing the vibration of the building with that of the neighboring ground in an earthquake, two horizontal pendulum seismographs of the same construction were used. One of them was set on the floor of a penthouse on the flat roof, and the other on the ground beneath a temporary shelter close to the building. The special feature of these instruments is that the record is taken with an open time scale. In earlier observations, the recording drums, which were driven by clockwork, had a peripheral speed of about 6 cm. per min., but it was soon found that, with such a condensed time scale, the motions could not be studied in detail. For this reason, the clockwork was replaced by an electric motor which ran continuously at such a rate as to make the time scale about 10 mm. per sec., in order to make such quick vibrations as 0.1 sec. clearly observable.

[12] *Bulletin*, Earthquake Research Inst., Vol. 7 (1929), No.1.

FIG. 53.—VIEW OF DESTRUCTION IN A VILLAGE IN THE PROVINCE OF IDU.

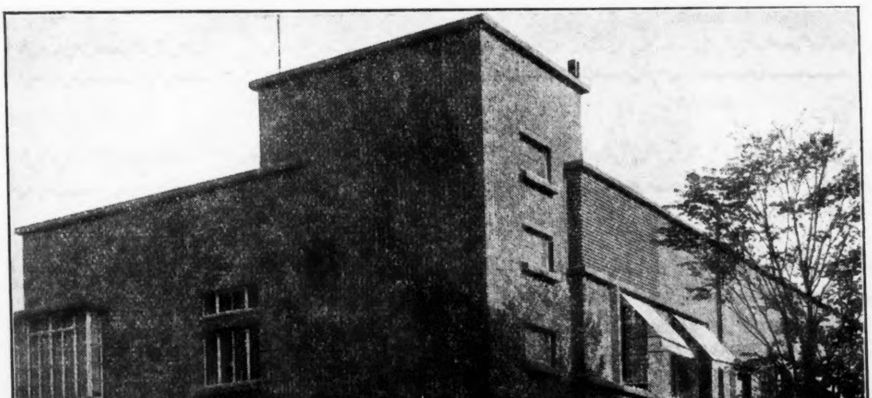


FIG. 54.—EARTHQUAKE RESEARCH INSTITUTE, TOKYO IMPERIAL UNIVERSITY.

Needless to say, an equally open diagram can be obtained by using a rapidly running drum driven by clockwork, the motion of which is started by a trigger arrangement; but, in that case, there will be difficulty in maintaining the same speed for the drums of the two instruments, a condition desirable for comparing the motions.

To simplify the mechanism, the recording drums were rotated in a fixed position without displacement in the direction of the axis. Therefore, it became necessary to keep the recording points clear of the smoked surface of the drum at ordinary times, lest it be marked for no purpose by a group of straight lines. To this end, an electric trigger was used to hold or release the recording pen according to necessity.

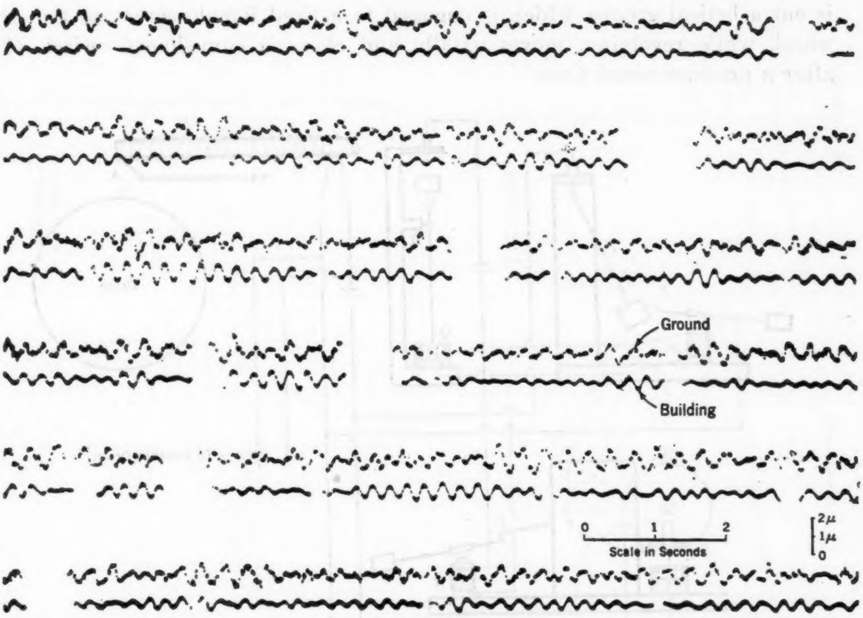


FIG. 55.—MICROTREMOR CURVE OF EARTHQUAKE RESEARCH INSTITUTE (A STEEL-FRAME, REINFORCED CONCRETE BUILDING) COMPARED WITH THAT OF THE ADJACENT GROUND.

As shown in Fig. 56, an electric current is ordinarily passed to the coil of a broad electric magnet (*N*) over the recording point (*A*) through a pendulum (*P*) and a catch-lever (*C*), under which another electric magnet (*M*) is located. Through the last-named electro-magnet (*M*) no current is passed at ordinary times, but when an earthquake occurs and sets in motion the trigger pendulum (*B*), then its point comes into contact with the mercury in an annular cup (*Q*), and an electric circuit is closed to excite the electro-magnet (*M*) and detach the catch (*C*). Thus, the electric current to the electro-magnet (*N*) is cut off to make the recording point fall on the surface of the drum, when it begins to record the earthquake motion. The recording point, however, must not be left in contact with the drum for an unduly long time, because then the useful portion of the record

is impaired by a series of unnecessary lines in the trail of the earthquake and after it is over. Obviously, an engineer does not need the whole of a seismic record. He only wants the most destructive part of it; the trail and the preliminary tremors are not very important.

For this reason, an arrangement is made to pull the recording point back again after a reasonable time (say, about 3 min.). For this purpose, a clockwork (*U*) is started by releasing another catch (*D*) by means of a magnetic relay (*R*), the electro-magnet of which works in parallel with that of (*M*) used for releasing the catch (*C*). Thus, as soon as the pendulum (*P*) is released, the clockwork is started and begins to revolve a spur wheel (*S*), which, in turn, drives another wheel (*W*). On the shaft of the wheel (*W*) is cut a helical groove, which is engaged to a fixed female screw, so that the wheel, while revolving, moves axially and closes a cam-shaped switch (*T*) after a predetermined time.

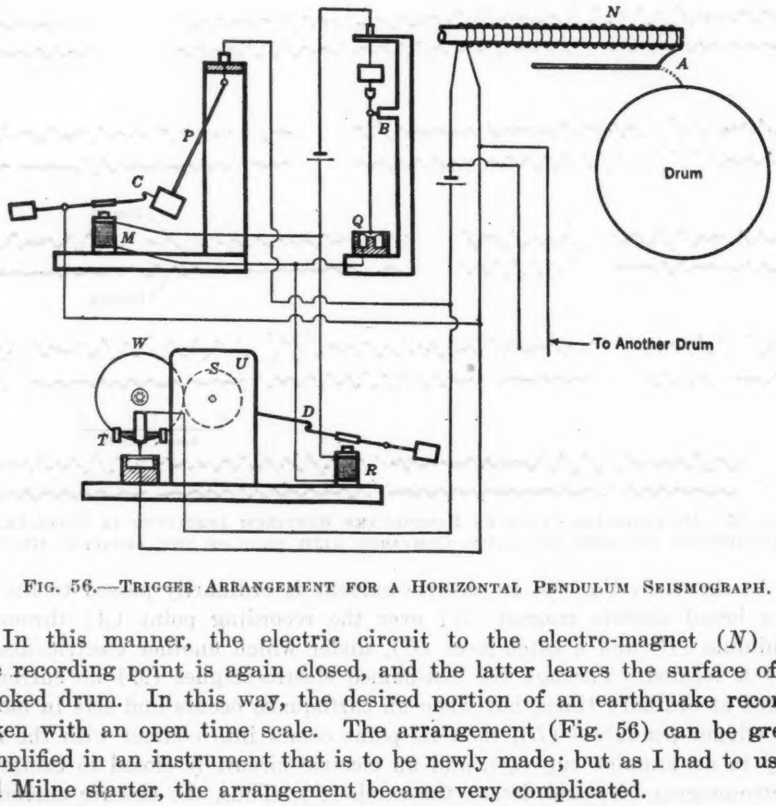


FIG. 56.—TRIGGER ARRANGEMENT FOR A HORIZONTAL PENDULUM SEISMOGRAPH.

In this manner, the electric circuit to the electro-magnet (*N*) over the recording point is again closed, and the latter leaves the surface of the smoked drum. In this way, the desired portion of an earthquake record is taken with an open time scale. The arrangement (Fig. 56) can be greatly simplified in an instrument that is to be newly made; but as I had to use an old Milne starter, the arrangement became very complicated.

Thus far, only one set of earthquake records have been taken, and that with partial success. This is shown in Fig. 57. Now, comparings the record of the motion on top of the building with that of the neighboring ground, two important facts are revealed: First, that so far as the general motion in

a moderate earthquake is concerned, a strongly constructed building like our Institute building, moves in just the same way as the surrounding ground; and, second, that the building is insensible to those vibrations of the ground having very short periods of, say, about 0.1 sec.

From these facts, it can be inferred that the dynamic stress induced in a strongly constructed rigid building by an earthquake is likely to be equal to the static stress which would be induced, had the building been subjected to

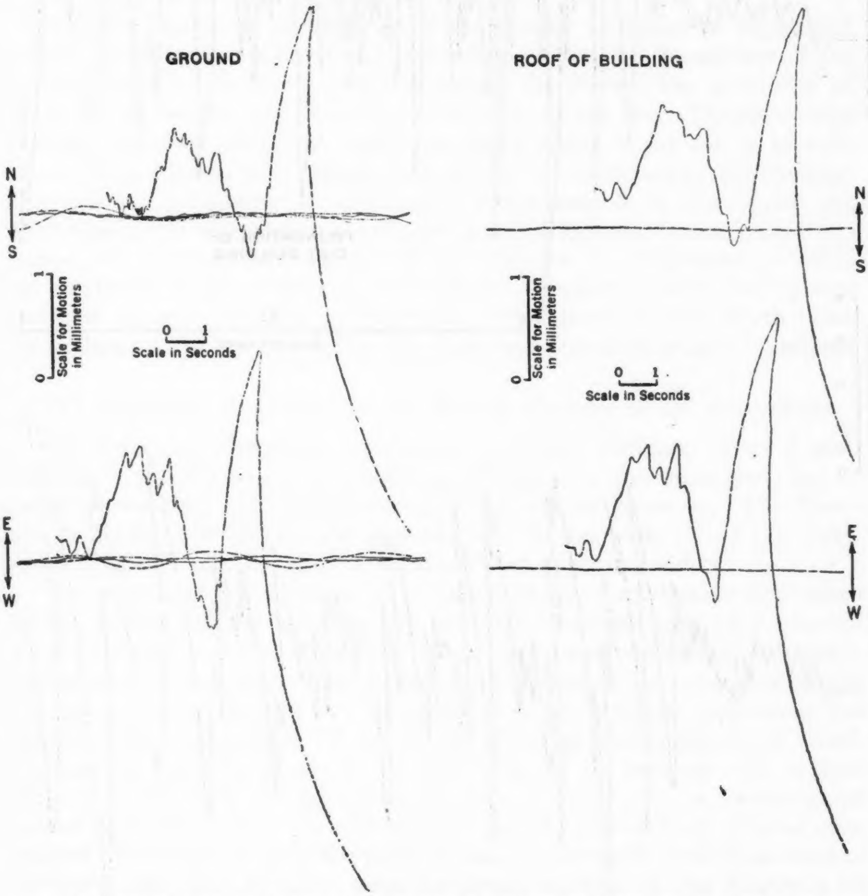


FIG. 57.—RECORDS OF AN EARTHQUAKE TAKEN NOVEMBER 26, 1930, ON THE ROOF OF THE INSTITUTE BUILDING, COMPARED WITH THOSE OF THE ADJACENT GROUND.

the static load of the intensity given by the mass of the building multiplied by the horizontal acceleration of the seismic vibration; and, also, that the component of seismic vibrations having a very short period, say, less than 0.1 sec., is not of much consequence for the building under consideration, although as stated in Lecture II, there is the possibility in some cases that such a quick vibration may cause a fairly intense acceleration.

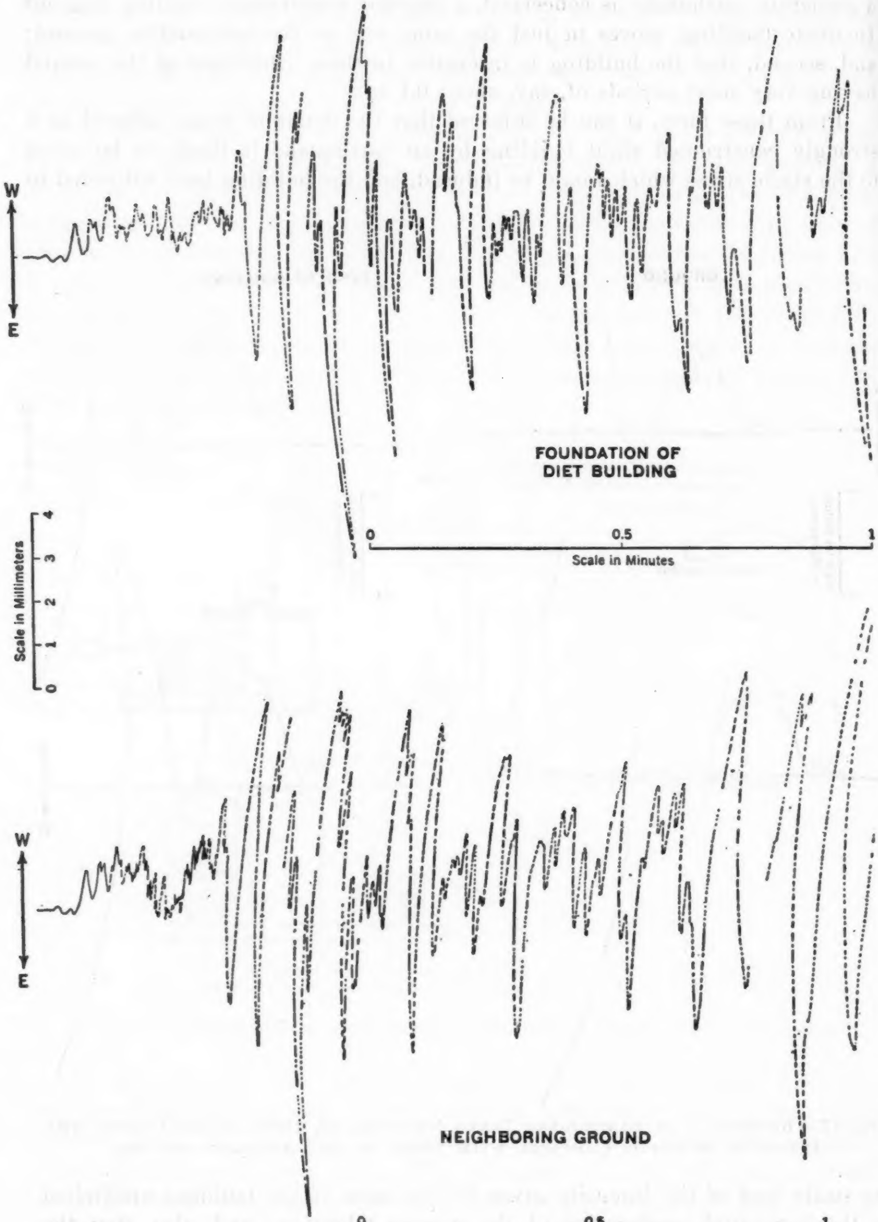


FIG. 58.—VIBRATIONS IN AN EARTHQUAKE IN THE DIET BUILDING FOUNDATION, COMPARED WITH THE ADJACENT GROUND DURING CONSTRUCTION.

(III) Motion of the Building Foundation

The last-mentioned fact would seem to be self-evident, since the natural period of oscillation of the building itself is greater than 0.1 sec.; but it is not the only evidence. According to observations made by Professor Imamura on the vibration of the Diet Building during construction, some very rapid ripples, having a period of about 0.1 sec., disappeared in the motion of the foundation, although the foundation moved about as much as the neighboring ground. One of the records is shown in Fig. 58.

Professor Ishimoto's investigation of the velocity of ripples on the ground is very useful in this connection. According to him, on the surface of the ground where our Institute building stands, the *P*-wave has a velocity of above 120 m. per sec. and the *S*-wave about 65 m. per sec. Therefore, very probably, the wave length of ripples having a period of 0.1 sec. is between 6.5 to 12.0 m.; hence, they are less than the linear dimensions of the building. Consequently, a building on soft ground is not sensible to those quick and short ripples. It may also be mentioned that this fact may be attributed to a certain extent to another behavior of the vibration of soft ground, in which the amplitude of the component of a seismic vibration of very short period decreases quickly with depth. Therefore, foundations at some depth below the surface will be less affected by the rapid components of seismic vibrations.

(IV) Vibration of a Building of Medium Rigidity in an Earthquake

The foregoing discussion refers to a very rigid building. Now I shall proceed to the discussion of a building belonging in the same class, but of moderate rigidity, and then to a weaker (non-rigid) structure. The Marunouchi Building which was strengthened after it was built (No. 1 (*c*), Table 3, and Fig. 47), may be taken as an example of medium rigidity.

The records of the vibration of this building in an earthquake were taken by Mr. T. Saita of our Institute. In this case, since the building is situated in the business center of Tokyo, no vacant space was available in the neighboring ground to install a seismograph. For this reason, one seismograph was placed on the floor of the basement, while an identical instrument was placed on the eighth floor. A set of records of an earthquake taken simultaneously at these two places are shown in Fig. 59. As the time scale of these records is not open (being nearly 60 mm. per min. for the seismograph placed high, and 65 mm. per min. for that placed in the cellar), detailed comparison of the motion, as in the previous case, is impossible with these records. Moreover, the usual period of main earthquake motions in this district is 0.7 sec., or more, and waves having shorter periods are superposed merely as secondary motions.

Since the period of exciting motion is thus generally greater than the natural period of the building, and since also the motion of the ground was recorded in the basement floor, instead of on the ground itself, the former of which would have more or less filtered or distorted the earthquake motion, comparison of these simultaneous records may be less instructive than the one previously mentioned. It is worth mentioning, however, that the general

motion at the top of the building is nearly the same as that in the basement, as regards its period, phase, and form, but the amplitudes are greater at the top by about 20 to 70%, depending on the nature of the motion; and on this general motion of the top of the building secondary motions having a period of about 0.5 sec., are superimposed, here and there. These secondary motions are likely to be the free vibrations of the building.

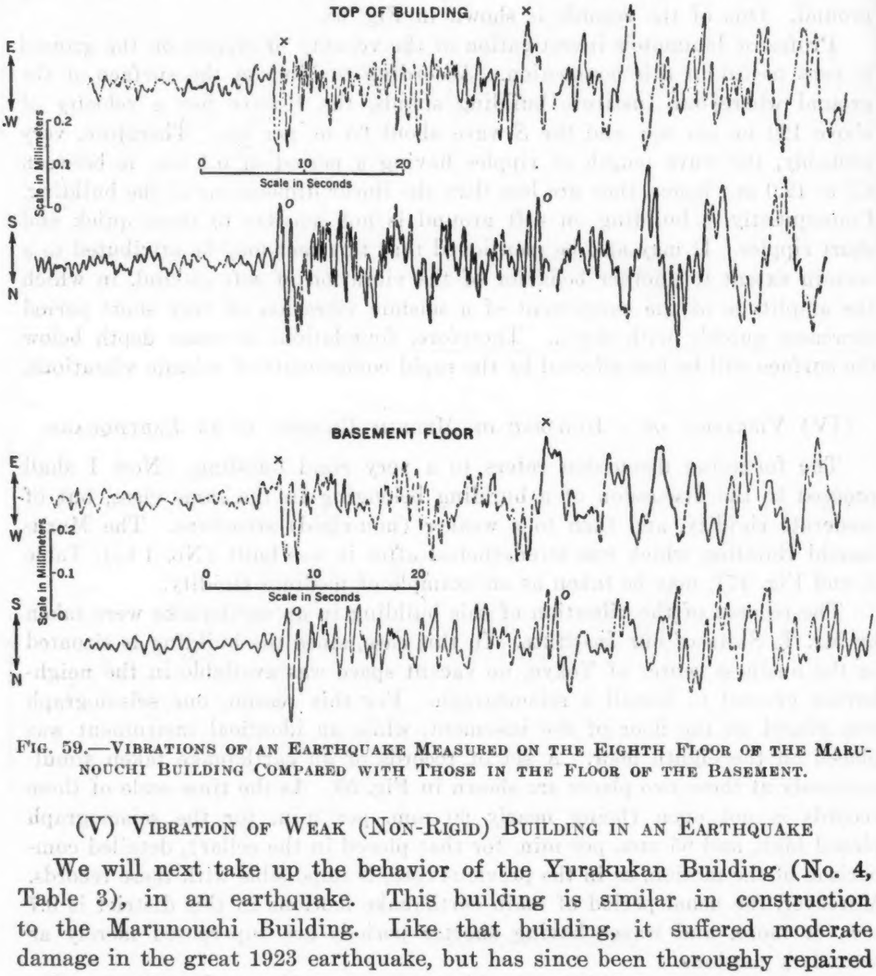


FIG. 59.—VIBRATIONS OF AN EARTHQUAKE MEASURED ON THE EIGHTH FLOOR OF THE MARUNOUCHI BUILDING COMPARED WITH THOSE IN THE FLOOR OF THE BASEMENT.

(V) VIBRATION OF WEAK (NON-RIGID) BUILDING IN AN EARTHQUAKE

We will next take up the behavior of the Yurakukan Building (No. 4, Table 3), in an earthquake. This building is similar in construction to the Marunouchi Building. Like that building, it suffered moderate damage in the great 1923 earthquake, but has since been thoroughly repaired and strengthened, so that at present its strength has quite come up to the standard. This is seen from the fact that the period of free vibration is now 0.5 sec. as against 0.81 sec. just after the great earthquake, and its vibration in an earthquake now is similar to that of the Marunouchi Building; but at the time the earthquake record was taken by Professor T. Taniguchi, of the Tokyo Engineering University (shown in Fig. 60), the

damage had not yet been repaired, so that the building was classed as weak. As was done in the previous case, one seismograph was installed on the basement floor and the other on the topmost floor.

As will be seen from Fig. 60, the nature of the vibration of this building in an earthquake is quite different from that for a rigid building and a building of medium rigidity. The motion of the basement floor which might not have been very different from that of the neighboring ground, was, as usual, very irregular, consisting of vibrations having periods varying from 0.7 sec. to 1.1 sec. On the other hand, the top of the building moved principally with a definite period of 1.0 sec., the amplitude of the maximum motion at the top being from two to three times that of the basement floor.

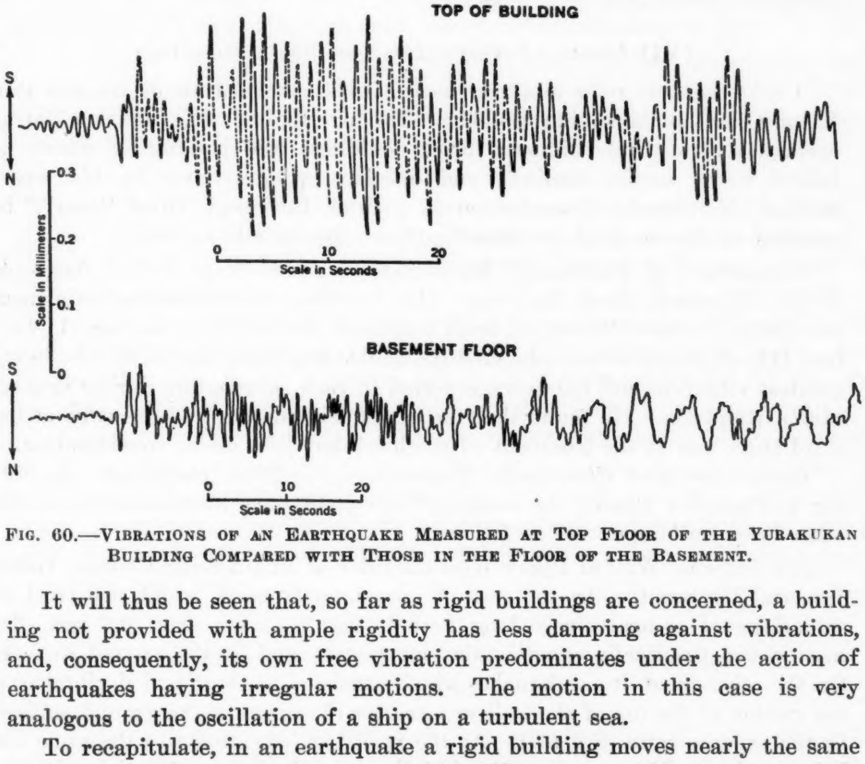


FIG. 60.—VIBRATIONS OF AN EARTHQUAKE MEASURED AT TOP FLOOR OF THE YURAKUKAN BUILDING COMPARED WITH THOSE IN THE FLOOR OF THE BASEMENT.

It will thus be seen that, so far as rigid buildings are concerned, a building not provided with ample rigidity has less damping against vibrations, and, consequently, its own free vibration predominates under the action of earthquakes having irregular motions. The motion in this case is very analogous to the oscillation of a ship on a turbulent sea.

To recapitulate, in an earthquake a rigid building moves nearly the same as the surrounding ground; a non-rigid building moves with its own period; and the motion of a building of medium rigidity is intermediate between these two.

By the way, I wish to say a few words about scientific researches on the behavior of structures in an earthquake. Generally, such researches—whether they are mathematical or experimental—are made under the assumption that earthquake motions are a continuation of one and the same simple harmonic motions. This is far from the truth. Still worse is the method in which only the established, forced vibration of structures is considered, little

attention being given to the free vibration and to the nature of damping against vibration. As every one knows, pure, forced vibrations are established only under the excitation of pure, harmonic motions and after a sufficient time —depending upon the resistance against vibrations—from the starting of the motion.

Such an argument as the foregoing will be unnecessary in view of what I have just mentioned of the actual behavior of structures in earthquakes. Also, it is a well-known fact among Japanese engineers that the fate of structures in a destructive earthquake is decided in the first 30 sec. of the principal motions, which is just contrary to the saying that the issue of a battle is decided in the last 5 min. These facts may be worth the attention of investigators of engineering seismology.

(VI) Omori's Investigations of Brick Buildings

I wish, here, to refer to a very useful investigation made by the late Professor Omori on a similar problem. He investigated the vibration during earthquakes, of three different brick buildings, the rigidity of which he judged to be ample, medium, and deficient, respectively. In his report entitled "Earthquake Measurement in a Brick Building; Third Paper",[13] he summed up the result of his investigations substantially as follows.

Comparison of Earthquake Movements at Third Story and at Basement of Old Mitsubishi Bank Building.—This building was constructed in a manner similar to those Mitsubishi brick buildings mentioned in Section (I) Lecture III. Professor Omori observed the double amplitude ($2a$) of the absolutely greatest vibration and the average period in each seismogram during twenty-nine earthquakes. He found that the earthquake motion was the same at the third story and at the basement of the brick building under consideration.

Comparison with Earthquake Measurements in Other Buildings.—According to Professor Omori, the results of the earthquake measurements in two other brick buildings were as follows:

(*a*) External Wall of Upper West Corridor of Engineering College, Tokyo Imperial University.—In the case of minor earthquakes, which consisted of vibrations of comparatively long period, that is, more than 0.5 sec., the motion was practically equal in the upper story and on the ground surface. On the other hand, in earthquakes which consisted of short-period vibrations, the motion at the top of the wall was greater than that of the ground surface in the average ratio of 2 to 1. As the period of vibration was the same for the two places of observation (this building and the Seismological Institute), it seemed to Professor Omori that in shocks of violent nature the wall, on which the roof rested, behaved like an inverted pendulum, its motion synchronizing with the earthquake motion.

(*b*) The Eastern End Wall of Natural History Museum of Tokyo Imperial University.—This is a two-story brick building situated not far from the College of Engineering. The total height of the wall is 53 ft., and the seismograph was attached to this wall at a height of 31 ft. from the ground.

[13] *Publications*, Earthquake Investigation Comm., No. 20, p. 73.

The wall, which was evidently weak, was shaken considerably by earthquakes, the duration and amplitude of motion being nearly three times those at the ground surface. In this case, the period of vibration of the wall was practically constant (according to Professor Omori), the mean value being 0.33 sec. This was taken to show that in an earthquake the wall acted as an elastic spring and vibrated with its own period, whatever the period and amplitude of the ground motion might be.

Commenting upon these examples, Professor Omori stated that they show,

> "* * * that the movements of brick buildings produced by earthquakes differ very much, according to the quality of the structures. In fact, the Natural History Museum, the Engineering College, and the Mitsubishi Bank may be taken as exemplifying respectively a very bad, an ordinary, and a good brick building, regarded from the seismological point of view."

In a poorly constructed building (according to Professor Omori), the walls are always shaken several times more severely at the top than at the foundations. An "ordinary" building in this category is one whose walls are shaken more severely than the foundation only in earthquakes of violent nature, or those consisting of vibrations of short period; and, finally, a "good" building is one whose walls have the same motion as the ground in all the earthquakes. Hence, a brick building such as the Mitsubishi Bank, which is shaken as a single unit by earthquakes, is to be regarded as having a great advantage in resisting earthquake shocks.

It will be very interesting to see just how the destructive 1923 earthquake has borne out Professor Omori's views. His conclusions were, indeed, confirmed with remarkable exactness, although, unfortunately, he did not live to see it. The Mitsubishi Bank Building (the "good" building according to Professor Omori), was perfectly intact; the Engineering College (the "ordinary" building), suffered more or less damage to gables, towers, coping stones, etc.; while the Natural History Museum (the "bad" building), suffered the most severe damage, although its walls remained standing. These may be valuable facts in the investigation of the earthquake-resisting quality of masonry buildings.

(VII) Damping Against the Vibration and the Natural Period of Beams

Theoretically speaking, the fact that one building is more effectively damped against vibrations than other buildings belonging to the same class and similarly built means nothing more than that its natural period is shorter than that of other buildings, because, as I have shown elsewhere[14], the "decay" of the free vibration of a beam is given by,

$$e^{-0.5kt} = e^{-b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, $b = \dfrac{0.5\,\xi}{E}\left(\dfrac{2\,\pi}{T}\right)^2$, and ξ is the coefficient of normal viscosity of

[14] *Bulletin*, Earthquake Research Inst., Vol. VI, p. 63.

the material composing the beam (in actual structures, the dissipation of vibrational energy due to other causes, such, for instance, as the dissipation through the foundation, etc., is to be included in this); E, the Young's modulus; T, the undamped natural period of the vibration of the beam; and e, the base of the Naperian logarithm. Evidently, the value of $\frac{\xi}{E}$ will be equal for similar buildings of the same material on the same ground, so that the "decay" constant, $\frac{k}{2}$, is inversely proportional to the square of the natural period of vibration.

Indeed, such a correlation between the damping and the natural period is seen in the studies of seismic vibrations of the steel-framed concrete buildings just described. Our Institute building, the most rigid of the three just mentioned, has the most effective damping, while its natural period is probably the shortest (the elastic vibration is not measurable). The Marunouchi Building, of medium rigidity, has a fairly large damping with intermediate natural period (0.5 sec.). Lastly, the Yurakukan Building, which was weak (non-rigid) at the time of observation, has the least damping and the longest period (0.8 sec.). As for brick buildings, Professor Omori has not given their natural periods, but very probably a relation similar to that just mentioned must have held among them.

(VIII) Strength of Buildings and Their Natural Periods

Now, the natural period of the transverse vibration of a beam is inversely proportional to the square foot of I, the geometrical moment of inertia of the section about its neutral axis, while both the induced stress in the bar and the deflection of the bar are inversely proportional to I. Therefore, it may be expected that, with similar buildings belonging to the same class, the shorter the natural period the stronger (more rigid) the building will be, apart from the question of resonance.

It may interest you to know how actual experience has shown that the natural period of a building decreases by increasing its rigidity. I will take the Marunouchi Building as an example. As will be seen from Table 3, and as already mentioned, this building had suffered damage from two earthquakes, and each time it was additionally strengthened the consequence being that the natural period became shorter with each repair. Originally, this building was constructed of comparatively light steel framing with curtain-walls of hollow tile, one and one-half bricks thick; the columns were covered with hollow tile and the partition walls were of hollow tile or metal lath, while the floors were of reinforced concrete without strong connections to the pillars. Thus, the steel frames were the only substantial members contributing to the rigidity of the building.

By the semi-destructive earthquake that occurred on April 26, 1922, one year before the great earthquake, the building suffered moderate damage, consisting of cracks and fissures in the curtain-wall and partition walls in the

second and third stories. The building was additionally strengthened as follows: (*a*) Most of the inside and outside pillars were strengthened by covering them with reinforced concrete and also by installing large knee-braces of the same material; (*b*) the principal partition walls were reconstructed of reinforced concrete; and (*c*) the diagonals and bracings were newly fitted between pillars on 172 panels, including the bearing partition walls.

By such additional strengthening, the original period of 0.9 sec. had been reduced to 0.7 sec. Then the great earthquake took place, resulting in similar damage as before. This second time the repairs were made more thoroughly, the principal work being as follows: (*d*) The old curtain-wall was entirely replaced by a reinforced concrete bearing wall; (*e*) the pillars were again additionally strengthened; and (*f*) 414 new reinforced concrete partition walls were added (some of them covering old bracings).

According to the estimate of the architect in charge, these alterations added about 20% to the weight of the building.[15] By this second strengthening the natural period was decreased to 0.5 sec. This actual example shows how the strength, or rigidity, of a building is intimately related to its natural period of vibration.

Thus, it will be seen that, in buildings similarly constructed and of nearly the same size, those having a shorter natural period have greater rigidity and more damping against vibration than those having a longer natural period. In other words, the former have more resistance against earthquake forces than the latter, provided the natural period of the building does not synchronize with that of the earthquake.

This view seems fully justified in the light of the 1923 great earthquake. It will be seen from Table 3, that Buildings No. 2(*a*) (Tokyo Kaikan), No. 3(*a*) (Yusen Building), No. 1(*c*) (Marunouchi Building), No. 6 (Kogyo Bank Building), and No. 5 (Kaijo Building), which are arranged in descending order of their natural periods, are nearly in descending order as to extent of damage suffered from the great earthquake.

(IX) Behavior of Buildings Constructed of Flexible Materials

One must not extend this inference too far for buildings constructed of flexible materials. Wooden-frame buildings constructed with reasonable care, and steel-frame structures without masonry walls are evidently highly resistant to earthquakes, the principal reason being that the ratio of the strength of these materials to their density is higher, and their ultimate elongation is greater than brick and concrete. Wooden buildings and bare steel structures, however, generally move with vibrations of their own period in an earthquake, just like weak masonry buildings.

For an explanation of this fact, let us again refer to Equation (3), and see how the "decay" constants differ with the various building materials. For this purpose, I have deduced the values of $\frac{\xi}{E}$ from various vibration

[15] For details, see report by Mr. T Saita in *Journal*, Inst., Japanese Archts., Vol. 41, No. 498.

experiments that were made with actual chimneys and frames conducted by several investigators, and have obtained their mean values as follows:

For masonry beams and frames:

$$3.2 \times 10^{-3} \text{ sec.}$$

For wooden-frame constructions:

$$1.5 \times 10^{-3} \text{ sec.}$$

For bare steel constructions:

$$5.8 \times 10^{-3} \text{ sec.}$$

(The last two values are not given in my paper cited previously[13], since they were calculated subsequently).

With regard to these values, it should be borne in mind that they are not physical constants giving pure, internal resistance, but practical constants covering the total dissipation of vibration energy from whatever cause it may be due. Therefore, they are more useful to us than pure, physical constants giving solid viscosity.

Now, for different beams having equal length, T varies as $\sqrt{\frac{\rho A}{EI}}$, in which, ρ is the density of the material composing the beam, A, the sectional area of the beam, and I, the geometrical moment of inertia of the section of the beam about its neutral axis. On the other hand σ, the induced stress due to accelerating motion perpendicular to the axis, is proportional to $\frac{\rho A z}{I}$, in which, z is the distance of the outermost fiber from the neutral axis. Substituting the latter relation for the former, we have,

$$T \propto \sqrt{\frac{\sigma}{E z}}$$

If we assume that the structural members in a building have strength complying with the Japanese Building Code, and also that the depths of the members are the same, we may take it that,

$$T \propto \sqrt{\frac{\sigma_0}{E}}$$

σ_0 being the stress allowed by the Code. Now the order of magnitude of $\frac{\sigma_0}{E}$ for different materials is as follows:

For concrete (compressive stress),

$$\frac{30 \ \frac{\text{kg.}}{\text{cm}^2.}}{1.6 \times 10^5 \ \frac{\text{kg.}}{\text{cm}^2.}} = 1.9 \times 10^{-4}$$

For pine,

$$\frac{65 \ \frac{\text{kg.}}{\text{cm}^2.}}{8.5 \times 10^4 \ \frac{\text{kg.}}{\text{cm}^2.}} = 7.6 \times 10^{-4}$$

For steel,

$$\frac{1\ 150 \quad \frac{\text{kg.}}{\text{cm}^2\text{.}}}{2 \times 10^6 \quad \text{cm}^2\text{.}} = 5.8 \times 10^{-}$$

Combining these values with those of $\frac{\xi}{E}$ previously given, the ratios of the decay constants given by Equation (3) are obtained as follows:

For concrete structure,

$$\frac{3.2 \times 10^{-3}}{1.9 \times 10^{-4}} = 17$$

For wooden structure,

$$\frac{1.5 \times 10^{-3}}{7.6 \times 10^{-4}} = 2$$

For steel structure,

$$\frac{5.8 \times 10^{-3}}{5.8 \times 10^{-4}} = 10$$

Thus, even under the arbitrary assumption that the depths of the members are equal, the damping against vibration of steel and wooden structures

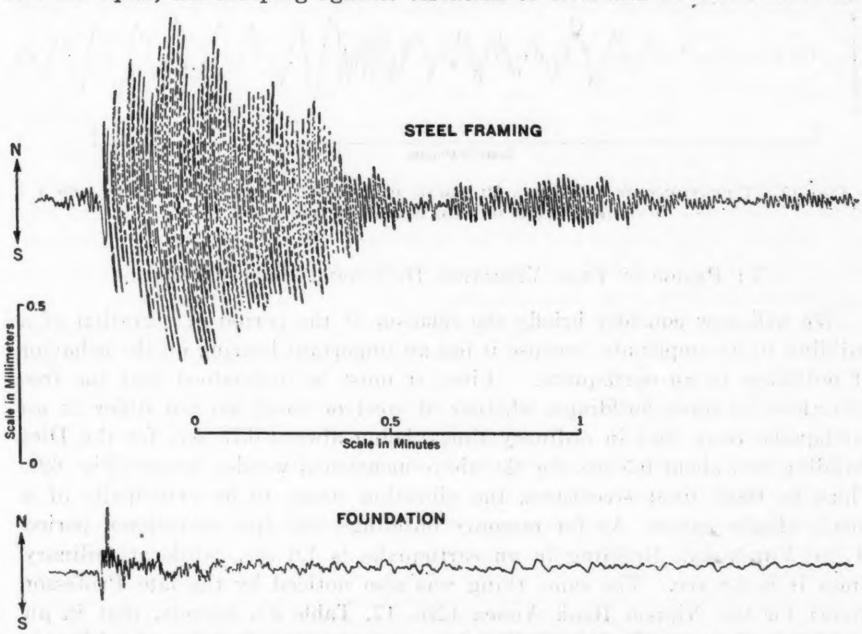


FIG. 61.—VIBRATIONS OF THE STEEL FRAME OF DIET BUILDING DURING AN EARTHQUAKE, COMPARED WITH VIBRATIONS OF THE FOUNDATIONS.

is less than that of masonry construction having the same earthquake-resisting strength. Fig. 61 shows the vibration of the steel framing of the Diet Building and the earthquake motion as measured by Professor Imamura and

Mr. F. Kishinoue, and Fig. 62 shows the same for a wooden building at Ito. This wooden building survived two destructive earthquakes. It will be seen that although both these buildings are strongly resistant to earthquakes, they vibrate remarkably during an earthquake with their own period. Therefore, with these types of buildings, the fact that they vibrate with their own period in an earthquake is no evidence of weakness.

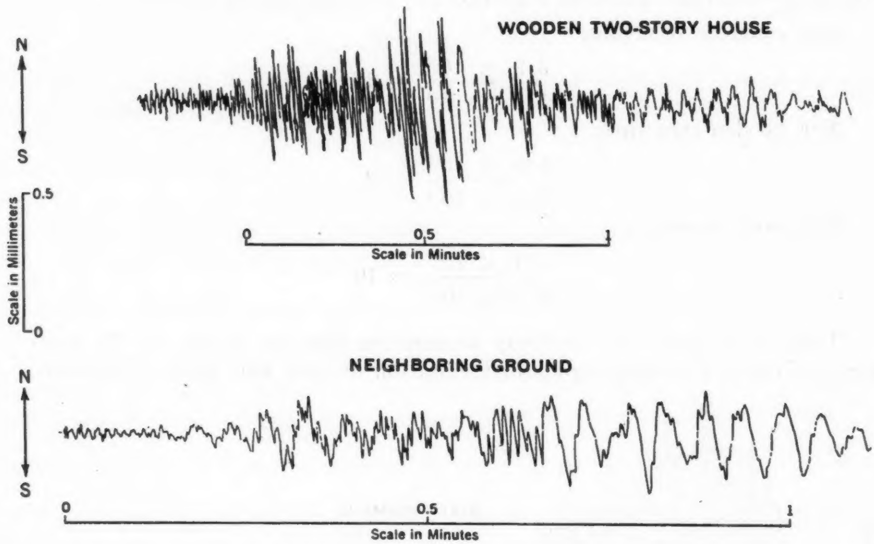


Fig. 62.—Vibrations of a Wooden Building at Ito During an Earthquake and a Seismograph of the Adjacent Ground.

(X) Period of Free Vibration Depending Upon Amplitude

We will now consider briefly the relation of the period of vibration of a building to its amplitude, because it has an important bearing on the behavior of buildings in an earthquake. First, it must be understood that the free vibrations of these buildings, whether of steel or wood, do not differ in an earthquake from that in ordinary times, being always 0.75 sec. for the Diet Building and about 0.5 sec. for the above-mentioned wooden house (Fig. 62). Thus, in these light structures, the vibration seems to be principally of a purely elastic nature. As for masonry buildings, the free vibrational period of the Yurakukan Building in an earthquake is 1.0 sec., while at ordinary times it is 0.8 sec. The same thing was also noticed by the late Professor Omori for the Nippon Bank Annex (No. 17, Table 3), namely, that in an earthquake the period of free vibration was somewhat lengthened, although not to such an extent as in the Yurakukan Building, the observed periods being 0.50 longitudinally and 0.46 transversely in an earthquake, as against 0.48 sec. and 0.43 sec., respectively, in ordinary times. I also observed it when measuring the microtremors of low reinforced concrete buildings in

ordinary times, namely, that the period of the microtremors depends more or less upon the amplitude of the tremor, being longer for larger amplitudes. This was reported in a paper to the Third Pan-Pacific Science Congress, 1926. This fact is also observable with our Institute Building (Fig. 55).

(XI) Yielding of Ground

Such a pseudo-harmonic nature of the vibration of masonry buildings may be attributed partly to imperfect elasticity of the material composing the buildings; but for small vibrations this effect is evidently too slight to be taken into consideration. Very probably its primary cause is the yielding of the ground-bed due to oscillation of the foundation of the masonry building, the weight of which is heavier than those built of wood or of bare steel framing.

As was described in my paper mentioned previously, the micro-vibrating motion of low monolithic buildings at ordinary times may be attributed to yielding of the ground-bed or of the rammed stone that lies beneath the footing of the foundation. It may be due partly to this yielding that, as previously mentioned, very rapid components of seismic waves cannot set a building into co-periodic vibrations.

It has also been observed that with a tall and narrow building, the earthquake motion recorded on the basement floor was intermingled with free vibrations of the building in a manner similar to those observed on high floors of weak but broad buildings. This shows that, like a rod, the foundation of a tall building is not "fixed", but more or less "free." Without the yielding of the ground such a motion is obviously impossible.

Such cushioning action of the ground at the time of an earthquake may serve more or less to relieve the destructive action of a strong earthquake in the case of masonry buildings. As has been mentioned, the 1923 Kwantō earthquake was more severe down town on the low oozy alluvial ground than on the high diluvial compact ground up town, so that the number of wooden houses overthrown down town was far greater than up town; but statistics of damage suffered by rigid masonry buildings for the two districts showed contrary results, as indicated in Table 4.

TABLE 4.—Percentage Ratios of Damaged Buildings to Their Total Number

	Wooden buildings *	Brick buildings †
High ground	3.7%	87.7%
Low ground	17.0%	81.4%

* From a report of Mr. G. Kitazawa, of the Metropolitan Police Board.

† From a report of Mr. K. Sato, of the Metropolitan Police Board.

As a matter of fact, those rigidly constructed buildings stood the earthquake comparatively better down town than up town. A similar condition was pointed out by the late Professor Milne in his book, "Seismology" (page 146), where he states that "Mallet, after his survey of the district devastated

by the Neapolitan earthquake in 1857, states that more places were destroyed upon the rock than upon loose clay or other materials," although Mallet attributed it to the fact that "there were more places situated upon the rock and hills than upon the alluvium and the plains." I cannot, of course, say whether or not, Mallet's explanation points to the real condition of things, but can merely remark that the phenomenon was quite the same as that which happened in Tokyo.

Besides these, the damage to buildings by the Northern Musashi earthquake which was mentioned in Section (IV), Lecture II, is of considerable importance in connection with the present question. As stated, this earthquake originated beneath a compact paleozoic region and then spread out to the alluvial plain of a large river. As may easily be imagined, the damage suffered by ordinary wooden houses was worse in the alluvial district than on the high paleozoic ground. On the other hand, rigid buildings like "dozos" (Japanese fireproof warehouses which are constructed of closely-spaced strong wooden pillars covered outside with thick plaster walls, and which stand on strong foundations), had their walls cracked to a greater extent on paleozoic than on alluvial ground.

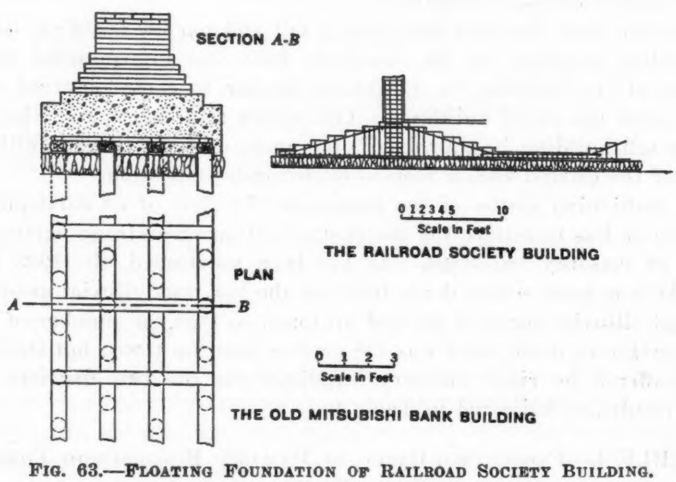


FIG. 63.—FLOATING FOUNDATION OF RAILROAD SOCIETY BUILDING.

Now, there is no doubt that the intensity of the 1923 earthquake was stronger down town where rigid buildings were less damaged. In my opinion, such an apparently paradoxical phenomenon was due to a certain extent, if not entirely, to the behavior of soft ground in an earthquake. A marshy alluvial ground could not exert more force on a building than the comparatively cohesive diluvial ground. There is, besides, the fact that the energy of elastic vibrations dissipates more quickly in the loosely fixed condition of a foundation on soft ground. It would more than counterbalance the stronger seismic motion down town. As a matter of fact, those buildings on soft ground with foundations consisting of one slab on rammed ground without any pilings (see Fig. 63) stood the shock far better than those having comparatively

strong individual footings resting upon deep pilings. It is also frequently observed after a severe earthquake that the soft ground surrounding a rigid building is permanently raised by the action of the earthquake, eloquent testimony to the fact that the foundation of the building did not move exactly as did the surrounding ground. These facts seem to support my views as to the behavior of soft ground in an earthquake. It may be worth the attention of architects and engineers.

(XII) The Most Destructive Part of Earthquake Motion

I should like to add a little more about the vibration of comparatively tall buildings at the time of the 1923 earthquake. As has been already mentioned, we have no data as to the motion of the earthquake in down-town Tokyo. In the circumstances, there is still more obscurity regarding the motion of the buildings in that district during that great earthquake; but the observation made by Professor T. Naito, of Waseda University, and by Mr. Y. Nagata, as Architect to the Metropolitan Police Board, on the number of broken electric pendant globes (all nearly 2 ft. 0 in. in length) in some of the office buildings, is very useful in that it indicates the mode of vibration of these buildings on that occasion.

According to their examinations, immediately after the earthquake, the percentage of broken electric light globes suspended beneath each floor was as shown in Table 5.

TABLE 5.—Percentage of Broken Electric Light Globes Observed After the 1923 Earthquake

Item No. corresponding to Table 3	Name of building	Number of Floor from Which Globes Were Suspended								Remarks
		Second	Third	Fourth	Fifth	Sixth	Seventh	Eighth	Ninth	
1 (c)	Marunouchi Building	..	94	99	99	100	99	98	99	† Ground floor
3 (a)	Yusen Building	4	82	85	85	85	85	64	..	* Is numbered
4 (a)	Yurakukan Building	16	43	55	56	63	64	30	..	* First
6	Kogyo Bank Building	27	44	23	44	50	58	44	..	*
	Kogyo Club Building	..	33	23	14	9	..	..	..	†

* T. Naito, Rept., Imperial Earthquake Investigating Comm., Vol. 100, c 1.

† Y. Nagata, Rept., Imperial Earthquake Investigating Comm., Vol. 100, c 2.

It will be seen that in all buildings other than the Marunouchi Building, the breakage in lamp globes was less on the highest and lowest floors than on the intermediate floors, suggesting that the middle parts of these buildings vibrated more than any other part. As to the Marunouchi Building, I cannot see why it behaved differently; perhaps the reason is that it moved so violently that even ahose globes in favorable positions were shaken so severely as to strike the ceilings.

Moreover, in all tall buildings that suffered more or less damage—for example, the Marunouchi Building (Fig. 47), the Yurakukan Building, the Yusen Building (Fig. 48), the Kaijo Building (Fig. 50), the Tokyo Kaikan (Fig. 49), etc., the damage that manifested itself as fissures and cracks

in walls, developed in the second and third stories. Thus, the damage resembled that usually suffered by chimneys, in which, on account of their having a natural period longer than that of the most destructive part of a violent earthquake, the damage occurs, as a rule, not at the base, but at some point above it.

These two facts seem to indicate that the earthquake motions which had the most detrimental effect on tall buildings were those components having periods shorter than the natural period of the building. This seems to endorse the view mentioned in Section (IV), Lecture II, that the predominant acceleration of an earthquake is generally due to secondary motions. It will be seen that such secondary waves manifest themselves here and there on the record which I have previously shown (Fig. 30), although, owing to the instrument being defective, the motion is not recorded as it really occurred.

(XIII) Underground Earthquake Motion

Lastly, a few words concerning the seismic motion underground may not be out of place, because it has an important bearing on the motion of a deeply seated foundation. Some seismologists and engineers seem to believe

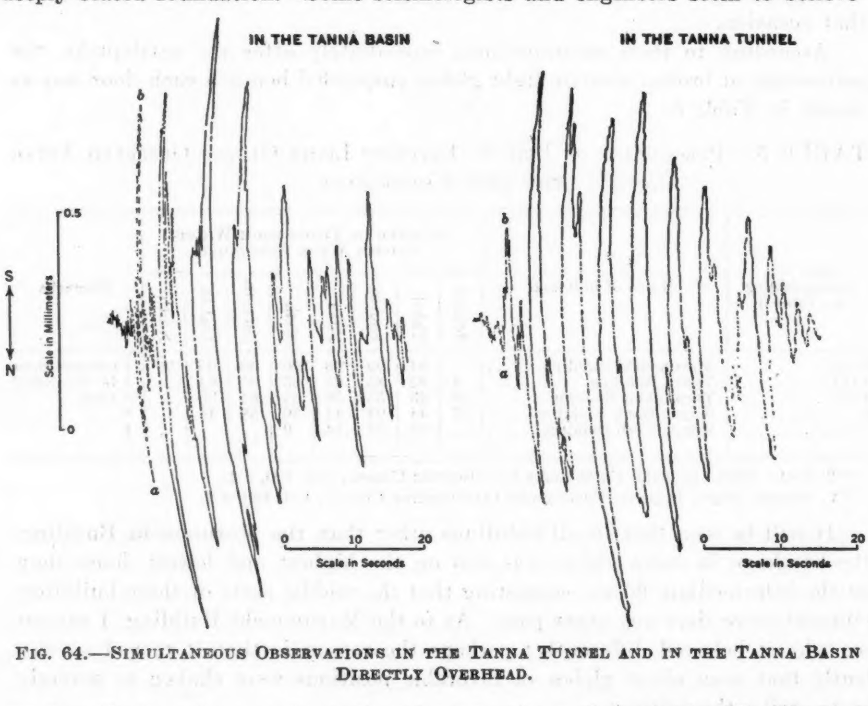


FIG. 64.—SIMULTANEOUS OBSERVATIONS IN THE TANNA TUNNEL AND IN THE TANNA BASIN DIRECTLY OVERHEAD.

profoundly in a decrease in earthquake intensity underground. The idea has perhaps originated from the result of a series of observations that were carried out by Professor Milne.[16]

[16] "A Seismic Survey Made in Tokyo," *Transactions*, Seismological Soc. of Japan, Vol. 10 (1887).

As mentioned in his paper, Professor Milne compared the seismic motion experienced at the bottom of a pit 10 ft. in depth with that experienced immediately above it, and found that the amplitude and the period of motion in the pit to those on the surface was as 1 to 3.4 and 1 to 0.65, respectively, thus making the ratio of maximum acceleration at these two places, 1 to 8.1; but it seems to me that there was something wrong in his observation. In our University there is an independent underground room for seismometric purposes, of nearly the same depth. Comparison of records taken on the floor of the underground room, with those on the surface, never showed such an enormous difference of motion as that observed by Professor Milne, although

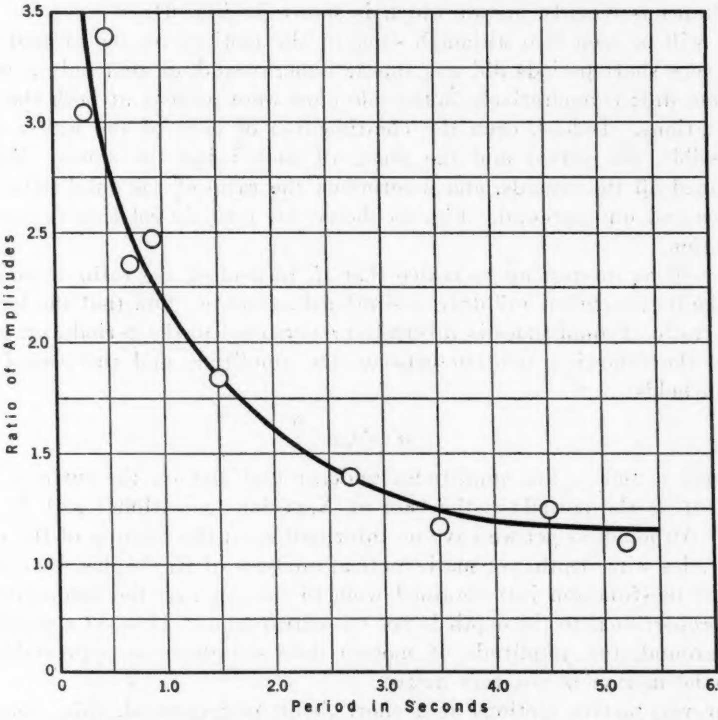


FIG. 65.—RELATION BETWEEN PERIODS OF MOTION AND RATIOS OF UNDERGROUND AND SURFACE AMPLITUDES.

a slight difference was observable. As can easily be imagined, this effect is only noticeable in vibrations having very short periods. For example, in vibrations having a period of about 0.2 sec., the amplitudes of motion in the underground room and on the ground surface vary nearly as 1 to 1.5. In vibrations having a period of about 0.4 sec., the ratio is 1 to 1.3, and so on. For vibrations having a moderate or long period (say, more than 0.8 sec.), no difference within the accuracy of ordinary seismographs is observable. At all events, the difference in periods mentioned by Professor Milne could never be observed. Generally, it is possible to identify each wave.

With regard to this matter, an important seismometric observation which we have been carrying out continuously since the occurrence of the Idu earthquake may be suggestive. As I have already mentioned, just after the occurrence of that earthquake, we installed two seismographs at Tanna, a village in the epicentral region; one on the surface of the Tanna Basin and the other inside the tunnel (Fig. 20), in a position 524 ft. perpendicularly below the former. Its purpose is to compare the earthquake motion underground with that on the ground surface, taking advantage of the probable occurrence of after-shocks. The seismographs in use are of the same construction and made of stainless steel, in order to withstand the corrosive effects in a damp place like a tunnel or a mine-pit. We have succeeded in taking a number of simultaneous records, one of which is shown in Fig. 64.

It will be seen that although some of the motions on the ground surface with very short periods did not appear underground, or that only a very few of them did; comparatively noticeable ones were present at both the observing stations. Indeed, even the identification of each of the waves was not impossible, the period and the phase of each being the same. Mr. Nasu examined all the records, and determined the ratio of the amplitudes on the surface and underground. Fig. 65 shows this ratio in relation to the period of motion.

It will be interesting to notice that if, instead of the ratio of amplitude, we take its logarithm and draw a similar diagram, we find that the logarithm of the ratio of amplitudes is inversely proportional to the periods, or, in other words, the following relation between the amplitude and the period of the motion holds:

$$a = a_0 e^{-\frac{K}{T}}$$

in which a and a_0 are amplitudes underground and on the surface, respectively; T is the period; e, the base of Naperian logarithms; and K, a constant. Although as yet we have no information on the relation of the ratio of amplitudes with depth, yet mathematical analysis of Rayleigh's wave and the form of the function just obtained seem to suggest that the assumption that K is proportional to the depth is not too extravagant. If so, at a small depth underground, the amplitude of motion does not decrease appreciably, provided the motion is not very active.

For very active motions at a short depth underground, this observation does not furnish us with useful data. As mentioned before, more or less decrease in amplitudes is certain in this case. Thus, in general, the decrease of amplitude underground is not so marked as stated in some textbooks. It appears, therefore, that the idea of constructing the footing of the foundation of a tall building, that has a long natural period, deeply underground for the purpose of reducing earthquake action is not as advantageous as believed by some engineers, although its advantages to a certain degree are unquestioned for a low building with a short natural period.

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